THE ELASTIC BEHAVIOUR OF EARTHQUAKE RESISTANT REINFORCED CONCRETE INTERIOR BEAM-COLUMN JOINTS

A report submitted in partial fulfilment
of the requirements for the Degree of
Master of Engineering at the University
of Canterbury, Christchurch, New Zealand.

by

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ABSTRACT

This report is concerned with the theoretical and experimental study of the behaviour of interior reinforced concrete beam-column joints under simulated earthquake loading.

An experimental program investigated the performance of two beam-column joint subassemblages subjected to static cyclic loading within elastic limits. The post-elastic behaviour of the two test units was then examined by testing to failure.

A theoretical method for analysis of the joint shear resisting mechanisms is reviewed and analyses of prototype beam-column joints are reported. Results of this analysis were then compared with those obtained from the test units. The design method is shown to provide a satisfactory and conservative estimate of the joint shear reinforcement required in an elastic beam-column joint.

The failure of the joints in the test units verified the expectations that their response to inelastic seismic load demands would have been unsatisfactory.

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NOTATION

A_{ch} = area of rectangular joint core measured to outside of hoop.

A_{cv} = effective area of joint core for shear resistance.

A_s = area of tension reinforcement.

A_s' = area of compression reinforcement.

A_{sc} = the lesser area of column flexural reinforcement at the tension or compressive force at a joint.

A_{sc}' = the greater area of column flexural reinforcement at the tension or compressive face at a joint.

A_{sh} = area of transverse hoop bar.

A_{sh}'' = cross-sectional area of hoop reinforcement including supplementary cross ties having a spacing s_h and crossing a section with dimension h".

A_v = area of shear reinforcement within a distance, s.

b_c = overall width of column.

b_j = effective joint width.

b_w = overall width of beam.

C_c = concrete compression force.

C_s = steel compression force.

d = distance from entrance compression fibre to centroid of tension reinforcement.

D_c = diagonal compression force in strut mechanism.

D_s = diagonal compression force in truss mechanism.

f_c' = specified compressive strength of concrete.

f_s = streel stress in tension face, or in bottom reinforcement.

f_s' = steel stress in compression reinforcement or top reinforcement.

f_y = yield strength of reinforcement.

f_{yh} = yield strength of horizontal joint shear reinforcement.

f_{yh}'' = yield strength of hoop reinforcement.

f_{yv} = yield strength of vertical joint shear reinforcement.

h_b = depth of beam.

h_c = depth of column.

h_{bj} = depth of joint core between centroids of beam reinforcement at the compression face and the tension face.

h_{cj} = depth of joint core between centroids of column reinforcement at the compression face and the tension face.

h'' = core dimension of tied column.

k_{col} = neutral axis depth factor for column section.

l_h = maximum unsupported length of rectangular hoop.

M_y^* = theoretical yield moment at critical beam section.

M_u = theoretical ultimate moment at critical beam section.

M_u^o = theoretical overstrength moment at critical beam section.

N_u = design axial load normal to cross section occurring simultaneously with V_u.

P_b = axial load strength at balanced load conditions.

P_e = observed beam load at which yield of beam tension reinforcement occurred, maximum design compressive load acting on column.

P_i = actual beam load applied.

P_o = ultimate load of axially loaded column.

P_y^* = theoretical beam load at which yield in beam tension reinforcement occurs.

s = spacing of shear reinforcement in direction parallel to longitudinal reinforcement.

s_h = centre to centre spacing of hoops.

T,T' = steel tension force.

v_c = nominal permissible shear stress carried by the concrete.

V_{ch} = horizontal joint shear resisted by concrete shear resisting mechanism.

V_{cv} = vertical joint shear force resisted by concrete shear resisting mechanism.

V_{jh} = total horizontal shear force across a joint.

V_{jv} = total vertical shear force across a joint.

V_{sh} = horizontal design joint shear force to be resisted by horizontal joint shear reinforcement.

V_{sv} = vertical design joint shear force to be resisted by vertical joint shear reinforcement.

v_u = nominal total design shear stress.

V_u = total applied shear force.

β = angle of inclination of diagonal strut,

ΔT_c = bond force transferred from beam steel to surrounding concrete within the diagonal strut.

ΔT_s = bond force transmitted from beam steel to the core concrete of the truss mechanism.

ρ = ratio of bottom reinforcement.

ρ' = ratio of top reinforcement.

ρ_b = reinforcement ratio producing balanced strain conditions.

ρ_s = ratio of volume of spiral reinforcement to total volume of core.

ρ_t = ratio of column reinforcement.

ϕ = capacity reduction factor.

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CHAPTER ONE

INTRODUCTION AND SCOPE OF REPORT

1.1 INTRODUCTION

According to present design philosophy for ductile reinforced concrete frames experiencing severe earthquake loading, there should be a desirable sequence in the formation of failure mechanisms in the structure. As it is difficult to evaluate the input forces into a reinforced concrete frame with any degree of certainty, the complete behaviour of the frame is unknown. Therefore designers attempt to ensure that a desirable ductile behaviour is attained. The concept of a 'weak beam - strong column' is encouraged as a means of enabling well distributed earthquake energy dissipation.

Present knowledge appears to ensure that a very high standard in the design and detailing of reinforced concrete beams and columns for strength and ductility can be achieved, and it is generally recognised that under severe seismic loads beam-column joints may become critical structural elements. The problem of the design of beam-column joints for seismic resistance has received considerable attention over recent years. Extensive tests both overseas and in New Zealand have been directed towards understanding the complex actions within the joint. This understanding is essential if the design of ductile reinforced concrete frames is to be done on a rational basis. Poorly designed joints which are liable to large strength and stiffness degradation under severe seismic load reversals could be expected to alter considerably the behaviour and safety of a reinforced concrete frame experiencing earthquake loading.

1.2 ISSUES OF JOINT DESIGN

Application of suggested design rules[1,2] often leads to congestion of reinforcement in the joint zone with a consequential difficulty in placing steel and concrete. It has been found that unless the flexural tension reinforcement content in the plastic hinge regions of beams is kept small, i.e. less than approximately 1.5%, the horizontal joint stirrup reinforcement may become so large that serious congestion of bars results.

Unsatisfactory joint behaviour, besides deteriorating shear strength, may be caused by slippage of beam bars within the joint due to a breakdown of bond.[4] The environment for bond in a joint core is liable to be adversely affected by the condition of the concrete as a result of extensive intersecting diagonal cracks and by yield penetration along the flexural bars of the beams into the joint from adjacent plastic hinges. Recent test specimens at the University of Canterbury, adequately reinforced for shear, have eventually failed by uncontrolled slip of the beam reinforcement.[7] This has led to limits being placed in New Zealand[11] on the diameter of beam bars passing through a joint. The extra number of bars required for bond control, coupled with increased joint reinforcement, has meant that some designers have had to increase member sizes to enable satisfactory steel placement in the joint.

More recent research has attempted to overcome these design problems and thus lead to less joint congestion. To avoid the problem of bond transfer, Fenwick and Irvine[8] have tested a beam-column joint with bond plates attached to the flexural steel which enabled transfer of joint shear forces, originating from the change in steel forces, from compression on one side to tension on the other, via a diagonal concrete strut.

Recent proposals[4,9] suggest that joints could be designed in such a way that the required energy dissipation occurs in potential plastic hinges of adjacent members, at a certain distance away, and not adjacent to the joint core region. This would enable a reduction in joint shear reinforcement which is necessary if brittle bond, shear or compression failures, accompanying significant inelastic deformations within the joint, are to be controlled. Relocated, potential plastic hinges, far enough away from the column face, should ensure that under reversed cyclic loading yield penetration will not enter into the joint. If the beam is correctly designed, the beam steel stresses at the support sections would be close to, but not above the yield level when the overstrength capacity of the critical section in the plastic hinge region is being developed.

1.3 THE AIMS OF THIS PROJECT

The main purpose of this study was to experimentally verify the validity of design proposals as set out in reference 9. These proposals suggest that when a plastic hinge is suitably located away from the column face, then, because there is insignificant inelastic action within the joint core, a substantial part of the joint shear can be carried by the concrete forming a diagonal strut in the core. This strut is assumed to extend between adjacent compression zones of the beams and columns of a plane frame.

Analytical design proposals for elastic joints formulated by Paulay, Park and Priestley in a recent study[13] are reviewed and appropriately extended as a result of the tests conducted.

The testing of a complete beam-column joint unit, including plastic hinges, was considered to be beyond the scope of this project. Current research at the University of Canterbury is investigating the problems involved in the design of relocated plastic hinges in the beams of reinforced concrete frames.

Two interior beam-column joint subassemblages were designed and tested so that the joint initially responded within its elastic limits. Results of these tests are reported and compared with theory. Subsequently the beam-column test units were loaded beyond the elastic limits till failure occurred.

1.4 THE SCOPE OF THIS PROJECT

Chapter one discusses the reasons for the testing program, and reviews present knowledge of the behaviour of beam-column joints under seismic type of loading. The problems associated with the design of beam-column joints to sustain inelastic deformations in adjacent members are discussed.

Chapter two presents the applicable code requirements and recommendations for beam-column joints under seismic load conditions.

Chapter three reviews a model for an elastic joint. This model postulates two major joint shear resisting mechanisms.

Chapter four describes the laboratory test program. The design, fabrication and instrumentation of the test units are reported. The loading sequence for the test units is discussed.

Chapter five presents the experimental results from the two interior beam-column joint units. These units were subjected to several elastic cycles before being loaded to failure.

Chapter six discusses the influence of a number of joint variables on the behaviour of an elastic beam-column joint. The results of several analyses are presented.

Chapter seven compares the results of the test program with those obtained from the theory. The theory is critically examined and areas of inadequacy are indicated.

Chapter eight draws conclusions from this research, and makes suggestions for future research.

1.5 ASSUMPTIONS USED IN THE THEORY OF JOINT BEHAVIOUR

Previous research[1,2,6] has identified the forces acting on an inelastic joint and the associated shear resisting mechanisms. To provide sufficient reserve strength within the joint, the forces in the beams and columns that meet at the joint in one plane are evaluated. In this, a factor which incorporates the effects of greater than specified steel strength, strain hardening of the flexural steel when beam hinges form, and other contributions to 'overstrength', such as slab reinforcing which may act integrally with the beam, is also considered.

The external forces in equilibrium at an interior joint are shown in Fig. 1.1.

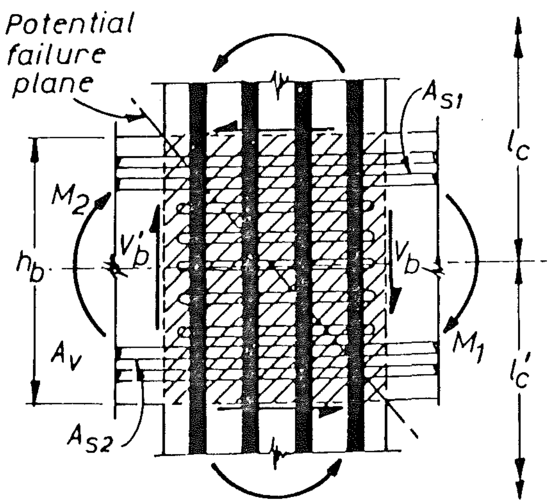


FIG. 1.1 SEISMIC ACTIONS IN EQUILIBRIUM AT A JOINT

The shear forces in the panel zone, as indicated in Fig. 1.2, are induced by the concentrated tension and compression forces from the adjacent beams and columns also considering the column and beam shear forces respectively. The horizontal shear force V_{jh} across the joint is from Fig. 1.2:

$$V_{jh} = A_{s1}\alpha f_y + A_{s2}\alpha f_y - V_{col} \tag{1.1}$$

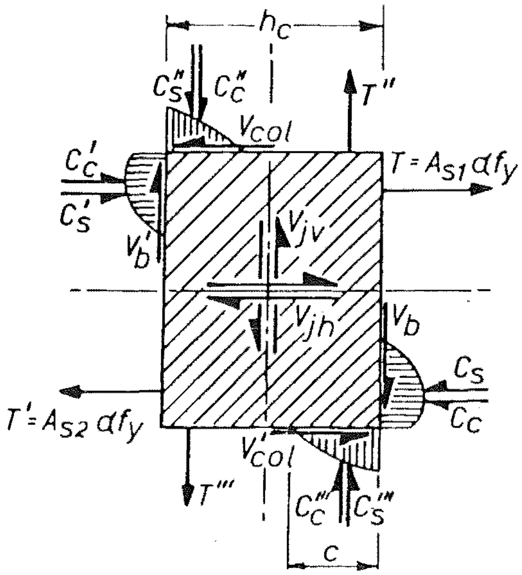


FIG. 1.2 INTERNAL CONCRETE AND STEEL FORCES AT A JOINT

The vertical shear force V_{jv} can be similarly evaluated from the internal column forces and the relevant beam shear force.

The shear failure plane in the joints of one-way frames has been observed to form along a diagonal from one corner of the joint to the other. With cyclic loading the diagonal tension cracks that form in the joint, open and close in each direction as the direction of load alternates. Upon yielding of the joint reinforcement, the cracks become wide and relative shear dislocation along the crack can lead to uneven bearing followed by grinding of the concrete and general deterioration of the concrete in the joint core. After one major excursion in each direction into the inelastic range of behaviour, the moment of resistance in potential beam hinges adjacent to the joint will be transferred entirely to the beam reinforcement. Permanent, large, full depth cracks develop across the plastic beam hinge and render the concrete ineffective in compression.

The shear transfer mechanisms in the panel zone may be idealised as due in varying proportions to: diagonal strut action, truss action, aggregate interlock, and dowel action. The diagonal compression force creates a splitting force perpendicular to it, and reinforcing steel is required to control the width of these cracks and to retain the strength of the concrete compression field. The forces induced in the panel zone by bond from the longitudinal reinforcement tend to be transferred by the truss mechanism comprised of a number of diagonal compression struts in the concrete, approximately parallel to the potential failure plane, and of tension ties in the horizontal and vertical planes. Usually, horizontal stirrup ties are provided to resist the horizontal forces. The vertical strut components must be resisted by intermediate bars, vertical stirrup ties, or special vertical bars. A contribution from aggregate interlock may be expected only where the cracks are narrow and the bearing surfaces are not worn. For this a sliding displacement along a diagonal failure plane is also necessary.

Dowel action in both the horizontal ties and the column bars would also contribute to shear transfer, although this would only be significant where the cracks are wide and the joint has deteriorated. The contribution of the numerous, large diameter column bars to horizontal shear transfer by the above mechanism could be significant under these circumstances.

CHAPTER TWO

CODE REQUIREMENTS

The following codes and recommendations are examined, being relevant to the seismic design of beam-column joints:

1. ACI 318-71 - Appendix A[3]
2. ACI-ASCE Committee 352 "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures".[1]
3. N.Z.S. 3101 P: 1970 "Provisional Standard for Reinforced Concrete Design". At present NZS 3101 P: 1970 is being revised, and proposals pertaining to beam-column joints have been formulated by a discussion group of the New Zealand National Society for Earthquake Engineering (NZNSEE).[9] These proposals are likely to be included in the new code.
4. P.W. 81/10/1 - May 1976 "Design of Public Buildings" draft revision.[12]

2.1 ACI 318-71 CODE - Appendix A ("Special Provisions for Seismic Design")

The ACI code considers eccentrically loaded ductile members in two categories. These are members with a design axial compression (1) less than 40 per cent of the balanced ultimate load (0.4 P_b) and (2) greater than 40 per cent of the balanced ultimate load.

If axial compression is greater than 0.4 P_b, the following requirements are applicable for confinement in columns:

(a) Hoop reinforcement shall be provided above and below connections for a distance equal to the overall depth of the member, 450 mm, or one sixth of the clear height of the column, whichever is the greatest.

(b) The hoop steel shall have a volumetric ratio, ρ_s, not less than

$$\rho_s = 0.45\{ \frac{A_g}{A_{ch}} - \} \frac{f_c'}{f_y} \tag{2.1}$$

or,

$$\rho_s = 0.12 \frac{f_c'}{f_y} \tag{2.2}$$

with $$A_{sh} = \frac{\ell_h \rho_s s_h}{2} \tag{2.3}$$

and the centre to centre spacing shall not exceed 100 mm.

If the axial load is smaller than 0.4 P_b, the column is required to be designed and detailed as a flexural member, satisfying the following requirements.

(a) Within a distance equal to four times the effective depth, d, from the end of the member, the amount of web reinforcement shall be not less than:

$$A_v \frac{d}{s} = 0.15\ A_s' \quad \text{or} \quad 0.15\ A_s\ , \tag{2.4}$$

whichever is the larger, and the spacing shall not exceed $\frac{d}{4}$.

(b) When longitudinal bars are required to act as compression reinforcement, stirrup ties spaced not further apart than 16 bar diameters or 300 mm are required. Such ties at column ends shall be provided for a distance of at least twice the effective depth, d, from the column base.

For all axially loaded members, transverse reinforcement in the columns must be provided to ensure that the shear capacity of the member is at least equal to the applied shears at the formation of the plastic hinge. The maximum spacing of shear reinforcement in columns shall be d/2.

The ACI 318-71 code suggests that the nominal shear stress to be carried by the concrete should be computed by:

$$v_c = 0.166\ (\ 1 + 0.073 \frac{N_u}{A_g}\ ) \sqrt{f_c'}\ \text{(MPa)} \tag{2.5}$$

However, v_c shall not exceed

$$v_c = 0.29\ \sqrt{f_c'}\ \sqrt{1 + 0.29 \frac{N_u}{A_g}}\ \text{(MPa)} \tag{2.6}$$

The remainder of the shear must be carried by hoops, such that

$$v_c + v_s = v_u \tag{2.7}$$

and

$$v_s = \frac{f_y A_{sh} d}{s} \tag{2.8}$$

Beam-column joints in ductile frames shall have transverse reinforcement proportioned from the above requirements with the design shear in the connection computed by an analysis taking into account the column shear and the shears developed from the yield forces in the beam reinforcement.

2.2 ACI-ASCE COMMITTEE 352

2.2.1 Joint Types

The recommendations for design of beam-column joints formulated by Joint Committee 352 are classified into two categories in accordance with the loading conditions for the joint:

Type 1: A joint for which the primary design criterion is strength and no significant inelastic deformations are expected.

Type 2: A joint connecting members for which the primary design criterion is sustained strength under reversals in the inelastic range.

The forces in the flexual reinforcement at the interface between a member and the joint shall be determined using the stress αf_y.

For type 1 $\alpha \geq 1.0$

For type 2 $\alpha \geq 1.25$

2.2.2 Strength Requirements

(a) Compression: The requirements for transverse reinforcement in the joint are similar to those of ACI 318-71. That is, if $P_u > 0.4 P_b$, then for confinement purposes the required area of rectangular hoop reinforcement shall be computed by

$$A''_{sh} \geq 0.3\, h''\, s_h \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f''_{yh}} \tag{2.9}$$

which is similar to equation (2.1).

Hoops shall be #3 (9.5 mm) bar minimum. For Type 1 joints, confined by members on all four faces of the column, or on two opposite faces, transverse reinforcement in the joint does not need to be provided in the direction of confinement, unless required for shear strength, development of reinforcement, or for confinement of bars in unconfined corners. The maximum spacing of ties in Type 1 joints which require transverse reinforcement is 150 mm.

For Type 2 joints, minimum transverse reinforcement is required such that,

$$\frac{A''_{sh}}{h'' s_h} \geq 0.12 \frac{f'_c}{f''_{yh}} \qquad (2.10)$$

The centre to centre spacing of rectangular ties for all Type 2 joints shall not exceed 100 mm.

(b) Shear: ACI-ASCE Committee 352 suggests that the nominal shear stress shall be computed on the horizontal plane by considering the horizontal shear forces on the boundaries of the joint and the horizontal normal forces generated by tensile and compressive forces in the beams framing into the joint, i.e. V_u. The nominal shear stress shall be computed by

$$v_u = \frac{V_u}{A_{cv}} \qquad (2.11)$$

where A_{cv} = the effective cross sectional area

The permissible shear stress carried by the concrete v_c in the joint shall not exceed the value given by the following equation

$$v_c \leq 0.29\, \beta\, \gamma \sqrt{f'_c \left(1 + 0.29 \frac{N_u}{A_g}\right)} \text{ (MPa)} \qquad (2.12)$$

(Note that for Type 2 joints it is recommended that N_u be taken as zero.)

Depending on the degree of confinement available perpendicular to the direction of shear forces being considered, that is if the members cover at least three-quarters of the width and three-quarters of the depth of the joint face, then γ shall be equal to 1.4 otherwise it shall be 1.0. For Type 1 joints β is taken as 1.4, and for Type 2 joints β = 1.0.

It is suggested that the nominal permissible shear stresses in the concrete may exceed those formulated by ACI 318-71 and given as equation (2.6), when unidirectional static loading is applied to members and only minimal ductility is required. Tests in which simulated seismic loads were applied indicated that a satisfactory estimate of the shear strength of the concrete is as given in equation (2.6).

Where the nominal shear stress, v_u, exceeds v_c, ties shall be provided so that

$$A_v = \frac{(v_u - v_c)\, A_{cv}\, s}{f_y d} \tag{2.13}$$

Shear reinforcement placed less than s/2 or 25 mm from the centroid of the concentrated tensile force generating shear shall not be considered effective.

For Type 2 joints the transverse reinforcement shall provide for not less than one third of the shear.

The value of $v_u - v_c$ shall not exceed $1.25\sqrt{f_c'}$ (MPa) and in no case shall v_u be greater than $1.66\sqrt{f_c'}$ (MPa).

2.3 NZNSEE RECOMMENDATIONS

Only beam-column joints affected by seismic actions are considered in these recommendations. The objective of the suggested design requirements is to make the joint stronger than the adjacent hinging members, and therefore to avoid significant inelastic behaviour within the joint core.

The design shear forces acting on a beam-column joint are to be evaluated from the maximum forces in all members acting at the joint at flexural overstrength of the hinging members.

2.3.1 Horizontal Joint Shear

It is suggested that the nominal horizontal shear stress in the joint v_{jh} should not exceed $1.5\sqrt{f_c'}$ (MPa), where

$$v_{jh} = \frac{V_{jh}}{b_j\, h_c} \tag{2.14}$$

with b_j taken as:

(a) when $b_c > b_w$

either $b_j = b_c$

or $b_j = b_w + 0.5\, h_c$, whichever is the smaller.

(b) when $b_c < b_w$

either $b_j = b_w$

or $b_j = b_c + 0.5\, h_c$, whichever is the smaller.

The horizontal design shear force to be resisted by the horizontal joint shear reinforcement should be

$$V_{sh} = V_{jh} - V_{ch} \tag{2.15}$$

where V_{ch} is the allowable horizontal shear force carried by the concrete shear resisting mechanism.

The value of V_{ch} should be assumed to be zero except in the following cases:

(a) When the minimum average compressive stress on the gross area of the column above the joint, including prestress where applicable, exceeds $0.1\ f_c'$

$$V_{ch} = 0.25\ (1 + \frac{f_c'}{25}) \sqrt{\frac{N_u}{A_g} - \frac{f_c'}{10}}\ (b_j h_c) \tag{2.16}$$

(b) When all beams at the joint are detailed so that the critical section of the plastic hinge is located at a distance of not less than the depth of the member or 500 mm away from the column face, then

$$V_{ch} \leq \frac{A_s}{A_s'} \frac{V_{jh}}{2} (1 + \frac{N_u}{0.6\ A_g f_c'}) \tag{2.17}$$

except that, where the axial column load results in tensile stresses over the gross concrete area, the value of V_{ch} should be linearly interpolated between the value given by equation (2.17) with N_u taken as zero, and zero when the axial tension stress is $0.2\ f_c'$. Thereafter the entire horizontal joint shear should be resisted by reinforcement.

The horizontal shear reinforcement should be capable of carrying the design shear force assigned to the reinforcement, V_{sh}, across a corner-to-corner diagonal tension crack plane. Therefore

$$A_{jh} \geq \frac{V_{sh}}{n\ f_{yh}} \tag{2.18}$$

where A_{jh} = cross sectional area of a set of multilegged horizontal stirrup ties, and

n = number of sets.

The required horizontal sets of stirrup ties should be placed between the outermost layers of the top and bottom beam reinforcement. A horizontal stirrup tie should be placed adjacent to each layer of beam flexural reinforcement, and other stirrup tie sets should be distributed uniformly within the depth of the joint core.

2.3.2 Vertical Joint Shear

The vertical design shear force to be resisted by the vertical joint shear reinforcement should be

$$V_{sv} = V_{jv} - V_{cv} \tag{2.19}$$

with

$$V_{cv} = \frac{A_{sc}}{A'_{sc}} \frac{V_{jv}}{2} \left(1 + \frac{N_u}{0.6\, A_g\, f'_c}\right) \tag{2.20}$$

Where axial tension stresses exist on the column then the value of V_{cv} is to be linearly interpolated in a similar fashion to that for equation (2.17).

The vertical joint shear reinforcement should consist of intermediate column bars, vertical stirrup ties, or special vertical bars placed in the column and adequately anchored to transmit the required tensile forces within the joint.

The effective area of vertical joint shear reinforcement should not be less than

$$\Sigma A_{jv} = \frac{V_{sv}}{f_{yv}} \tag{2.21}$$

The spacing of column bars in each plane of any beams framing into a joint should not exceed 200 mm, and in no case should there be less than one intermediate bar in each side of the column in that plane.

2.3.3 Confinement

The horizontal transverse confinement reinforcement in beam-column joints should not be less than that contained in reference 10, except that where the joint is adequately confined by beams on all four column faces, with no potential plastic beam hinges at the column face, this can be modified. In no case should the stirrup tie spacing in the joint core exceed ten times the diameter of the column bar or 150 mm, whichever is less.

A summary of the applicable recommendations from reference 10 are given below.

The total area of hoop bars and supplementary cross ties when $P_e \leq 0.6\ f_c'\ A_g$ should be not less than

$$A_{sh} = 0.3\ s_h\ h''\ [\frac{A_g}{A_c} - 1]\ \frac{f_c'}{f_{yh}}\ [0.33 + 1.67\ \frac{P_e}{f_c'\ A_g}] \tag{2.22}$$

or

$$A_{sh} = 0.12\ s_h\ h''\ \frac{f_c'}{f_{yh}}\ [0.33 + 1.67\ \frac{P_e}{f_c'\ A_g}] \tag{2.23}$$

whichever is greater.

The minimum tie diameter is required to be not less than 8 mm. The supplementary cross ties and legs of hoops should not be spaced transversely more than either 200 mm or one-quarter of the column section dimension perpendicular to the direction of the transverse steel.

Each longitudinal column bar should be laterally supported by the corner of a hoop having an included angle of not more than 135^o, or by a supplementary cross tie, except where the distance between two laterally supported bars does not exceed 200 mm between centres.

The yield force of the hoop bar or supplementary cross tie should be at least one-sixteenth of the yield force of the bars it is to restrain.

The spacing of hoop sets shall not exceed the smaller of

(a) one-fifth of the smaller column section dimension

(b) 150 mm

(c) six times the diameter of the longitudinal bar to be restrained.

2.4 "DESIGN OF PUBLIC BUILDINGS", PW 81/10/1

The design recommendations contained in this code of practice tend to be stricter than those contained in the other codes and recommendations listed. The suggested design procedure is similar to that of reference 2 for seismic resistant joints, but a point of interest are the recommendations for limits on size of bar.

To guard against a premature bond failure and slip of flexural steel within the joint, it is recommended that for:

Beams	$d_b < h_c/25$	$f_y = 275$ MPa
Columns	$d_b < h_b/25$	$f_y = 275$ MPa
"	$d_b < h_b/35$	$f_y = 380$ MPa

where d_b = diameter of bar,

h_c = depth of column member in direction of shear, and

h_b = depth of beam member in direction of shear.

2.5 SUMMARY OF RECOMMENDATIONS

The main differences in approach of the above recommendations are summarised. For the purpose of joint design, the NZNSEE recommendations suggest vertical joint shear should be considered as well as horizontal joint shear. A method for determining the proportion of the shear force resisted by the concrete in both directions is given.

The ACI-ASCE 352 and the ACI 318-71 approach, ignores vertical shear and gives no indication that vertical shear reinforcement may be required through the joint. ACI-ASCE 352 suggest that the design axial load shall be taken as zero for Type 2 joints as the axial load is influenced by overturning forces and vertical accelerations. The NZSNSEE recommendations presented here also cover the evaluation of column forces in reference 11 which indicates how axial load effects should include the influence of probable beam overstrength and possible magnification of column moments due to dynamic effects. This reference provides methods for such evaluation. The ACI-ASCE 352 approach in this respect would appear to be conservative, especially in the case of interior joints of large multistorey frames where net axial tension is improbable.

ACI-ASCE 352 recommend that for Type 1 and Type 2 joints the shear stress carried by the concrete may be increased by 40%, where sufficient confinement is provided by members perpendicular to the direction of the shear force. Whereas NZNSEE recommendations are that, with potential plastic beam hinges near the column face, confinement from the transverse members may not be relied on.

For confinement, the NZNSEE approach is that the amount of transverse reinforcement required for a given section is dependent on the axial load on the section. ACI-ASCE 352 suggest that for any given section, the transverse reinforcement required should be independent of the axial load.

CHAPTER THREE

ELASTIC JOINT MODEL

3.1 INTRODUCTION

This chapter reviews the analytical design proposals presented by Paulay, Park and Priestley,[13] for elastic joints in a ductile reinforced concrete frame experiencing severe seismic loading. The internal concrete and steel forces at such an 'elastic' joint are shown in Fig. 3.1 at the end of the chapter. The horizontal shear force V_{jh} across the joint is:

$$V_{jh} = A_s f_s + A_s' f_s' + C_c - V_{col} \tag{3.1}$$

First it will be assumed that there is no axial load acting on the column. The internal concrete compression forces, together with the column and beam shears and bond forces, could form a system in equilibrium within the joint core. This concrete mechanisms is shown in Fig. 3.2. The principal component of the concrete mechanism is a diagonal concrete strut, which transfers the force D_c between the corners of the joint core, and ΔT_c is the component of the total bond force to be transferred within the joint core by the diagonal concrete strut. This is expected to occur in the regions of transverse compression supplied by the internal concrete forces.

As these internal concrete forces represent a significant proportion of both the horizontal and vertical shear forces across the joint, it is evident that a large part of the joint shear may be carried by the above mechanism. It is postulated that the shear resistance of mechanisms associated with aggregate interlock forces along the diagonal cracks and those with dowel shear across the reinforcement passing through the joint are insignificant compared with the mechanism shown in Fig. 3.2.

The possibility of aggregate interlock being significant has been mentioned by some researchers based not on measurements but on known beam action. But the system of forces in equilibrium at a joint differs substantially from that occurring in beam action. Dowel action is associated with larger shear displacements than those expected to occur in an elastic joint, therefore, it is considered that this mechanism should be neglected.

By considering the concrete forces in equilibrium at the lower right-hand corner of the joint in Fig. 3.2, the horizontal component of the diagonal compression force, D_c, can be defined as

$$V_{ch} = C_c + \Delta T_c - V_{col} = D_c \cos \beta \qquad (3.2)$$

where ΔT_c is the bond force transferred from the beam steel to the surrounding concrete within the shaded area of the strut. With the remaining steel forces in equilibrium, large bond forces may be introduced into the joint core. These bond forces will impart shear stresses to the core concrete.

In most cases the diagonal tension capacity of the concrete in the joint would be exceeded at relatively small loads as the tensile splitting strength of the concrete is normally less than 10% of its compressive strength. As the loads were increased it would be expected that the resistance of the concrete against the shear forces introduced by the longitudinal beam and column bars would break down.

If the joint core is suitably reinforced, with effectively anchored horizontal and vertical steel, a truss mechanism can be developed in which the confined core concrete supplies the necessary diagonal compression field with a capacity of D_s. This truss mechanism with the steel forces introduced by the beam and column bars is shown in Fig. 3.3. With reference to Fig. 3.3, the part played by the tension and compression members of the truss mechanism in resisting the shear on the edge of the panel can be readily seen. It should be noted that for this model with no vertical axial load acting, horizontal shear reinforcement alone is not sufficient. A joint so reinforced does not satisfy the basic requirements of equilibrium.

To maintain a diagonal compression field, such as in Fig. 3.3, horizontal and vertical compression forces at the joint core boundaries are required. Some codes[1] ignore this concept of vertical equilibrium and treat the problem of shear in beam-column joints as one of horizontal equilibrium only. By viewing the maintenance of the diagonal compression as a problem of horizontal and vertical equilibrium it follows that either:

(a) Distributed horizontal and vertical reinforcement effectively anchored at or beyond the boundaries of the joint core is required, or that

(b) External compression forces, such as gravity compression on columns or central prestressing in beams, is required.

A practical solution is to use horizontal stirrup ties and distributed vertical column bars placed so that they pass through the joint core. It is emphasised that the vertical compression force applied to the joint core by vertical joint reinforcement and compression load on a column is as essential as the horizontal stirrup tie reinforcement if the truss mechanism (Fig. 3.3) is to function.

It is convenient to denote the horizontal shear resistance of this mechanism by

$$V_{sh} = V_{jh} - V_{ch} = \Delta T_s = D_s \cos \beta \qquad (3.3)$$

where $\Delta T_s = C_s + T' - \Delta T_c$ (Fig. 3.3), is a bond force transmitted from the beam reinforcement to the core concrete of the truss mechanism. Similarly the truss mechanism will sustain a vertical shear force V_{sv}.

From considerations of equilibrium and the recognition of a potential diagonal failure plane across the joint, as shown in Fig. 1.1, it is evident that horizontal shear reinforcement needs to be provided so that

$$A_{jh} \geq \frac{V_{sh}}{nf_y} \qquad (3.4)$$

where n is the number of sets of multilegged stirrup ties, with a cross sectional area of A_{jh}, that are uniformly distributed in the joint core between the top and bottom beam reinforcement.

The vertical joint steel reinforcement should be capable of sustaining, in addition to tensile loads that may be transmitted to it from the columns, a tensile force of

$$V_{sv} = D_s \sin \beta \qquad (3.5)$$

3.2 THE ALLOCATION OF SHEAR STRENGTH TO THE CONCRETE AND SHEAR STEEL RESISTING MECHANISMS

For design purposes the relative proportion of the horizontal shear, V_{jh}, that is resisted by the concrete mechanism, V_{ch} (Fig. 3.2) and by the truss mechanism, V_{sh}, (Fig. 3.3) is required. The concrete and steel shear mechanisms are assumed to be additive so that

$$V_{jh} = V_{ch} + V_{sh} \tag{3.6}$$

To illustrate the relative magnitudes of the shear resisting mechanisms based on the proposed concepts, a vector diagram (Fig. 3.4) is used. The simple equilibrium requirements for the elastic joint are also indicated on this diagram. With reference to Fig. 3.1, the internal concrete and steel forces can be determined by elastic theory. It is assumed for simplicity in this example that there is equal top and bottom beam reinforcement provided (i.e. $A_{s1} = A_{s2}$). The maximum steel stresses are assumed to be close to but not exceeding yield, and the resultant horizontal forces (Fig. 3.4) are those acting at the level of the bottom reinforcement.

The ability of the joint to allow satisfactory bond transfer has been recognised as an important aspect of joint performance. A rational assumption of the bond stress distributions along bars passing through the joint needs to be made. For the elastic joint a linear steel stress variation and a corresponding uniform bond force distribution, u, is assumed (Fig. 3.5). The cover concrete over the column bars on the tension face is assumed to be unable to absorb transverse tensile stresses.

A part (ΔT_c) of the total steel force ($T + C_s$) (Fig. 3.4) will be transmitted to the diagonal strut of the concrete shear resisting mechanism. It (ΔT_c) combines with the concrete compression force C_c and the column shear V_{col}, to develop, together with similar vertical internal column forcees, the principal diagonal compression force D_c (Fig. 3.2). The remainder of the total horizontal steel force ΔT_s will be part of the truss mechanism, shown in Fig. 3.3, which, when combined with corresponding vertical bond forces from the column reinforcement, will give rise to D_s.

Fig. 3.4 combines these mechanisms and shows realistic relative proportions of all forces discussed above. Previous research has indicated that with negligible axial load on the column the concrete shear resisting mechanism could account for over one half of the total joint shear. The total diagonal force, $D = D_c + D_s$, remains constant and proportional to the total joint shear to be resisted. As the joint is elastic, also under cyclic loading, little degradation in the ability of either mechanism to carry shear is expected. The relative proportions of D_c and D_s should, however, change according to the level of axial load acting on the joint.

3.3 THE STRENGTH OF THE COMPRESSION FIELD

For the concrete to act as a satisfactory shear resisting mechanism, limits on the diagonal compression to be carried are necessary. This is to safeguard against a premature and possibly sudden brittle compression failure of the concrete. The concrete struts that form are bounded by diagonal cracks in the core. They are subject to complex loading and distortions which would mean that the normal crushing strength of the concrete could not be attained. When the diagonal cracks form, the ties crossing the cracks have tensile strains induced in them. As a consequence the tie imposes transverse tension on the concrete strut bounded by two parallel cracks. The strut is thus subjected to biaxial tension and compression. This is known to reduce the unidirectional compressive strength of the concrete.[2]

Limits on the value of the nominal joint shear stress are normally specified by codes [1,3] to guard against overload of the compression field. The appropriate limit for an elastic joint is unknown at this stage. The elastic nature of the joint should enhance the ability of the cracks formed under cyclic loading to close and bear more evenly than in the corresponding inelastic case. Thus it would be expected that the allowable nominal joint shear stress for elastic joints could be higher than that applicable to joints behaving inelastically.

3.4 THE EFFECT OF AXIAL LOAD

Axial compression is expected to increase the shear strength of a beam-column joint. The simple mechanisms explained previously can be extended to explain how axial compressive column load contributes to shear resistance.

Fig. 3.1 shows (with dashed lines) that as a result of vertical compression load on the column the neutral axis depth at the boundary of the joint will increase to c^*. As a consequence a larger proportion of the development of beam bars will be in the zone of transverse compression (Fig. 3.5). Equilibrium considerations require that the main diagonal compression force D_c^* becomes steeper and that it engages an appropriate horizontal force $(C_c + \Delta T_c^* - V_{col})$ to maintain its inclination β^*. Thus the share of the horizontal steel force ΔT_c^*, that will combine with the total column compression stresses, must become larger. Fig. 3.6 shows qualitatively the distribution of the horizontal joint shear components V_{ch}^* and V_{sh}^* with axial compression P_u, while exactly the same beam moments are applied as in the previous example, shown in Fig. 3.4. Thus it can be seen that for the same beam moment input, the higher the axial load the less joint shear reinforcement will be required.

3.5 ADVANTAGES OF AN ELASTIC JOINT

Several advantages become apparent from this discussion on the elastic joint model.

(a) As steel stresses at the boundaries of the joint do not exceed yield, concrete strains are limited and hence the concrete compression stresses are relatively low.

(b) With inelastic behaviour adjacent to the joint, permanent, large, full depth cracks develop across the plastic beam hinge near the column face and render the concrete ineffective in compression due to plastic elongation of the flexural reinforcing. The moment of resistance in the beam hinges for the most part would be provided only by the forces in the reinforcing steel. With an elastic joint the concrete forces should not substantially diminish with cyclic reversed loading as all tension cracks should close upon load reversal.

(c) As seismic loading is instantaneous, a substantial proportion of the flexural compression force can be expected to be transmitted by the concrete with no significant redistribution due to creep.

(d) As tensile yielding cannot occur, yield penetration, interfering with the development of the required elastic strength of the flexural reinforcement and efficient bond transfer, cannot take place. Therefore the effective anchorage length of the beam bars is likely to be maintained with more favourable bond conditions.

(e) The concrete compression forces, with an appropriate proportion of the bond forces from the reinforcement passing through the joint, can combine to form a linear arch, D_c, similar to that shown in Fig. 3.4. Due to the elastic nature of the joint the effectiveness of this arch would not be expected to deteriorate even with cyclic reversed loading. The horizontal component of this arch, V_{ch}, which in part resists the total horizontal joint shear V_{jh}, could be maintained.

(f) As a corollary to the above point, a smaller shear force, V_{sh}, need be allocated to the truss mechanism. This means there would be a substantial reduction in the joint shear reinforcement required compared with an inelastic joint, with a consequential lessening of steel congestion in the joint core.

Analyses of elastic joints have indicated that when there is no axial compression on the column, and equal top and bottom beam flexural reinforcement is used, a little less than one half of the horizontal joint shear need be carried by joint reinforcement, i.e. $V_{sh} < 0.5\ V_{jh}$. It should be remembered, however, that the contribution of other shear resisting mechanisms, that of dowel action of the vertical column reinforcement in particular, has not been considered in such analyses.

(g) In view of improved bond conditions in the joint region and absence of yield penetration along the beam bars, larger size beam bars could be used. Consequently a reduction in the number of bars is possible. As the shear strength of such a joint is greater, the flexural reinforcing content in the beams could be increased, thereby enabling the use of shallower members.

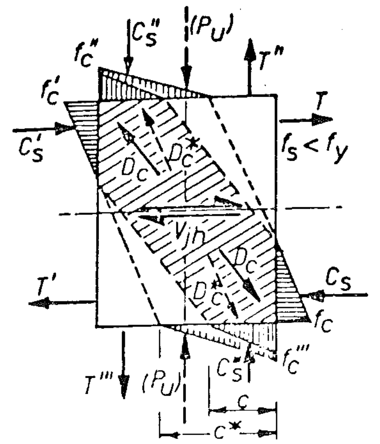


FIG. 3.1 INTERNAL FORCES AT AN ELASTIC JOINT

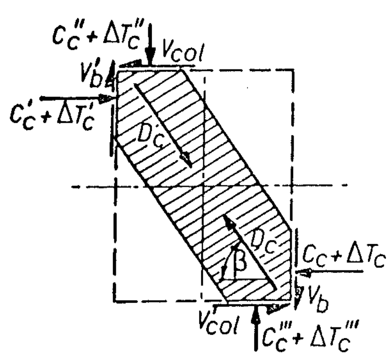


FIG. 3.2 CONCRETE MECHANISM

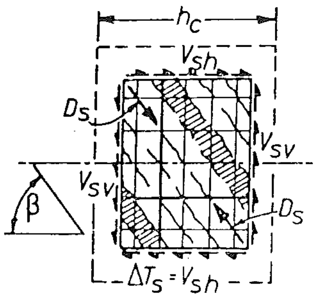


FIG. 3.3 TRUSS MECHANISM

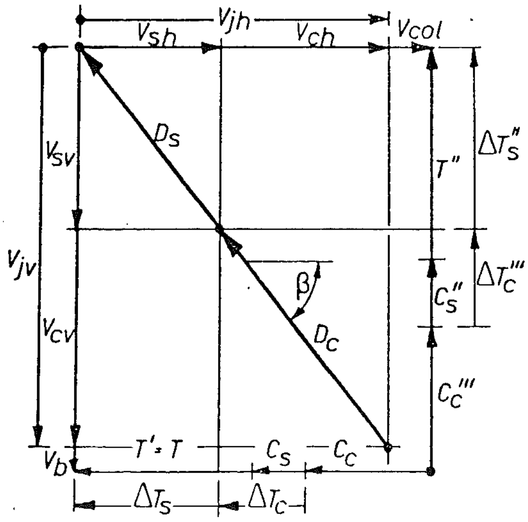


FIG. 3.4 INTERNAL FORCES AND COMPONENTS OF JOINT SHEAR

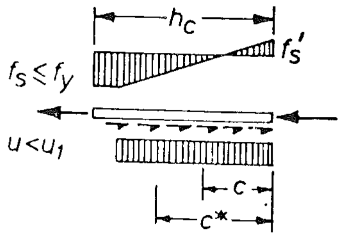


FIG. 3.5 STEEL AND BOND STRESS DISTRIBUTIONS

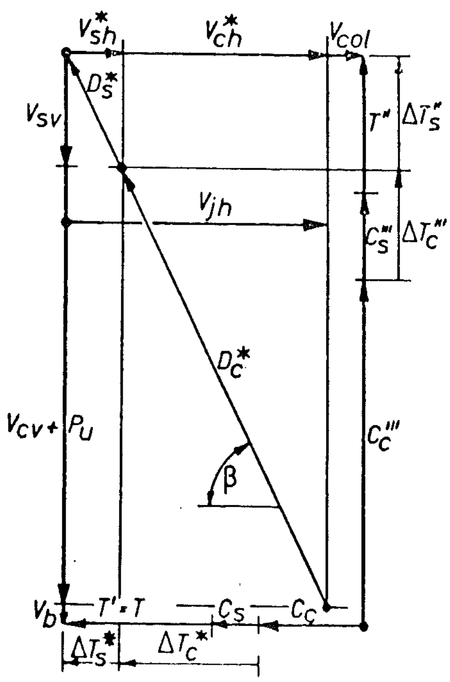


FIG. 3.6 INTERNAL FORCES AND COMPONENTS OF JOINT SHEAR WITH AXIAL COMPRESSION

CHAPTER FOUR

TEST PROGRAM

4.1 INTRODUCTION

The specimens of this study represented an interior beam-column joint subassemblage with the beams in one plane only. The specimens were close to full size with beam dimensions 610mm × 356mm and column dimensions of 457mm × 457mm. Fig. 4.1 gives the overall size of the specimens with the applied loads and other relevant information.

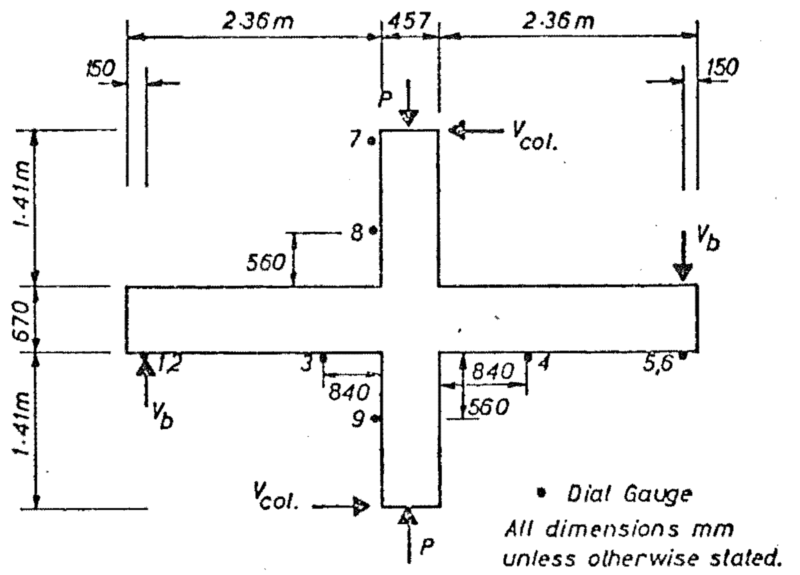


FIG. 4.1 TEST SPECIMEN

Two previous specimens tested at the University of Canterbury by Beckingsale[7] formed the basis of the design of units B1, with low axial load, and B2, with high axial load, enabling comparisons to be made with respect to their relative performance.

The intent of the design for units B1 and B2 was that the beam flexural steel should be stressed to less than or equal to yield stress when the applied horizontal joint shear force is the same as that occurring at the ultimate load in Beckingsale's unit B12. The ultimate load in unit B12 occurred after the formation of plastic hinges in both beams at the column face. To enable beam hinges to form and to prevent significant inelastic deformation within the joint, Beckingsale used a considerable quantity of joint reinforcement. This was more than that suggested by ACI-ASCE Committee 352.[1] The overall specimen performance to be reported by Beckingsale, was somewhat compromised by the slippage of the beam steel.

As previously stated, a properly detailed plastic hinge away from the column face can ensure that:

1. The beam flexural steel close to the joint is stressed below yield when inelastic deformations are imposed on the structure, thus reducing the danger of bar slip in the joint.

2. With the inelastic deformations restricted to the hinge region, the joint should largely deform within its elastic limits, provided that there is no substantial spread of yield from the hinge region.

Beckingsale's units B12 and B13 had identical beam flexural steel contents, i.e. $\rho = \rho' = 0.0086 = 0.175\ \rho_b$. An axial load of 311 kN (= 0.046 P_o) was applied to unit B12, and 8 sets of 4 legs of 12.7 mm joint ties were provided assuming that they would carry the entire horizontal design joint shear force. Unit B13 carried initially a column load of 2890 kN (= 1.32 P_b = 0.43 P_o) and contained only 6 sets of 4 legs of 12.7 mm ties for joint reinforcement. This reduction in joint shear reinforcement was made to take into account the increased capacity of the concrete to carry shear under high axial load.

Unit B12 was loaded cyclically and with stepwise increases a maximum displacement ductility of six was attained. The joint core maintained its integrity in the test, and plastic hinges formed in the beams near the column face. Slip of the beam steel caused severe stiffness degradation when large displacement were applied, but little load degradation was observed.

Unit B13 was loaded to D.F. 6 at a column load of 2890 kN (0.43 P_o) and as the specimen performed well, the axial load was subsequently lowered to 1680 kN (0.25 P_o) with further inelastic load cycles being applied up to a displacement ductility of 6. The lowering of axial load, although accompanied by larger stiffness deterioration, did not affect the good performance of the specimen.

Detailed calculations for the test specimens B1 and B2 are given in Appendix A. However, a summary of the reinforcement used is given in the following sections.

4.2 THE BEAMS OF UNITS B1 AND B2

The theoretical ultimate beam moment (M_u^*), based on a steel couple, for Beckingsale's unit B12 was 256 kN-m. The observed ultimate moment applied during the test was 321 kN-m. This was based on the average of ultimate beam tip loads applied to both beams. This corresponds with an overstrength factor ϕ_o of 1.25, due to strain hardening of beam steal.

It was intended that units B1 and B2 should have a maximum applied moment, similar in magnitude to the moment at overstrength (M_u^o) in Beckingsale's unit B12. The moment at the elastic limit (M_y^*) was defined as the moment causing yield of the tension reinforcement at the column face. An elastic analysis of the beam section indicated that 8-D20 reinforcing bars ($\rho = \rho' = 0.013$) were required with M_y^* equal to 334 kN-m. Table 4.1 indicates the beam reinforcement used in units B12 and units B1, B2 with the theoretical moment values at yield and ultimate. The observed overstrength moment in unit B12 and the theoretical overstrength moment for unit B1 are also shown.

Table 4.1 A Comparison of Main Beam Reinforcement and Beam Moments

	Unit B12 6 # 6	Units B1, B2 8 - D20
A_s (mm^2)	1710	2513
f_y (MPa)	298	288
M_y^* (kN-m)	237	334
M_u^* (kN-m)	256	347
M_u^o (kN-m)	321[1]	434

[1] observed value.

It was decided, for reasons explained in section 4.5, that a moment of 288 kN-m, being the average of M_u^* and M_u^o for unit B12, should be applied at the column face in a cyclic manner.

The shear reinforcement in the beam was the same as that used in unit B12. The arrangement of beam steel is shown in Fig. 4.2.

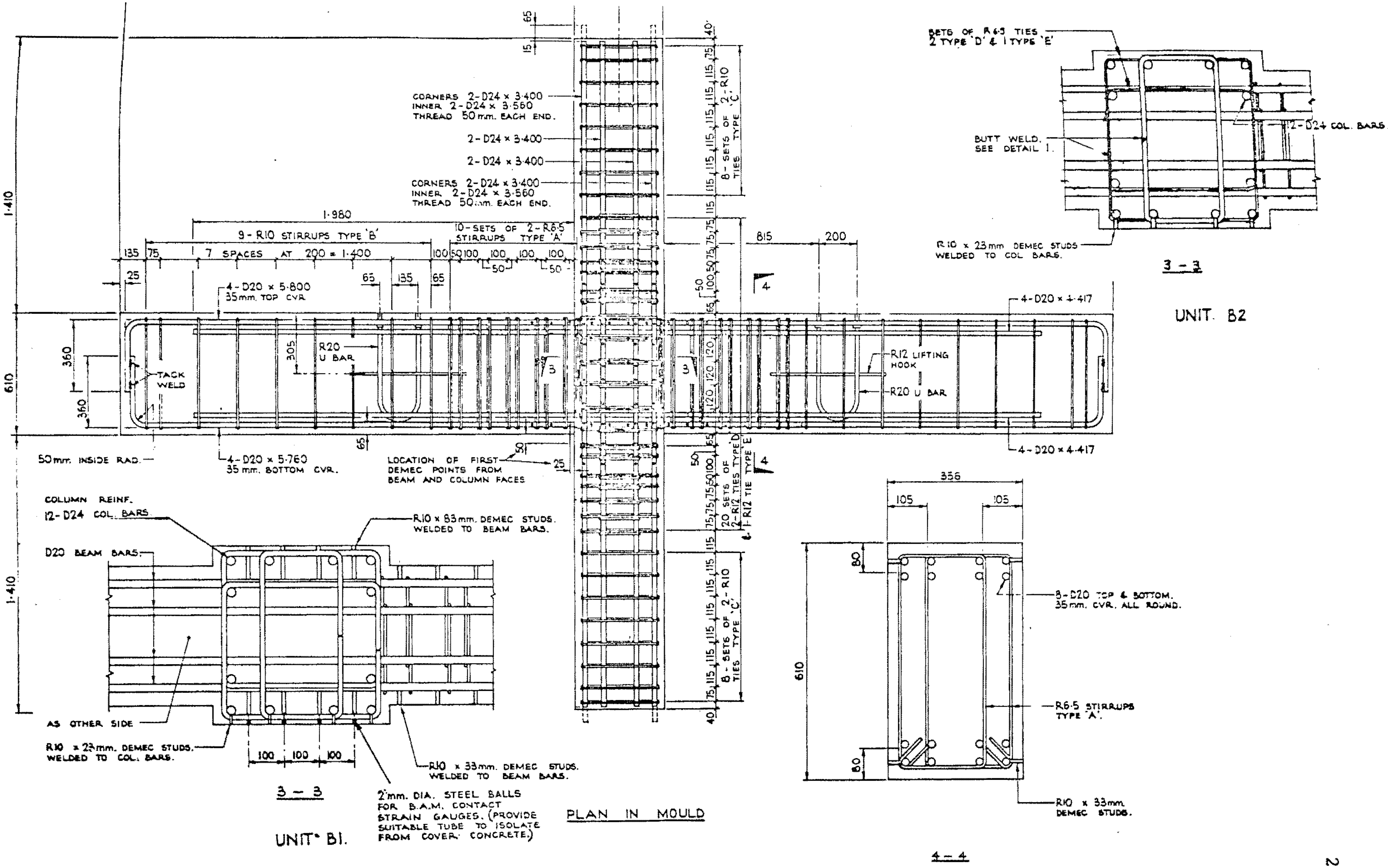


FIG. 4.2 REINFORCING DETAILS

4.3 COLUMNS OF UNITS B1 AND B2

The columns of units B12 and B13 were reinforced with 12#7 (22.3 mm) HY60 reinforcing bars (ρ_t = 0.022). The column was designed so that there was sufficient reserve strength to avoid plastic column hinges forming before those in the beams.

As there were no #7 (22.3 mm) bars available at the time of testing units B1 and B2, it was decided to use D24 bars. The nominal yield strength of this steel was 380 MPa. The same steel layout as in unit B12 was also adopted for units B1 and B2. The ultimate moment capacity of the columns was checked at an axial compression load of 311 kN for unit B1 and 2890 kN for unit B2. The design column moment was derived using an overstrength factor ϕ_o = 1.25 for the beam moment input even though it was not intended that beam hinges with substantial strain hardening of beam steel would occur. The ideal moment capacity ($\phi = 1.0$) of the column of unit B1 was 1.14 times that required, and for unit B2 it was 1.07 times that required. In normal seismic design of ductile frames where beam hinging is desirable, the ideal column flexural capacity is usually at least 50% larger than that of the beams framing into it.

As the longitudinal column steel was considered to be part of the vertical joint shear reinforcement, it was decided that this joint reinforcement should not be increased much in excess of what would be required for this purpose. This avoided the creation of possibly more favourable joint conditions than those that existed in Beckingsale's test. The chosen vertical column reinforcement was thus very similar to that used by Beckingsale.

Table 4.2 Main Column Reinforcement

	Unit B12 12#7	Units B1,B2 12 - D24
A_s (mm^2)	4655	5429
ρ_t	0.022	0.026
f_y (MPa)	427	427

Detailed calculations for the column sections are given in Appendix A.2.

4.4 JOINT

4.4.1 Horizontal Shear Reinforcement

The joint shear reinforcement was proportioned by calculations based on the elastic model outlined in Chapter three. The internal beam and column forces at the joint faces were obtained using elastic theory. The horizontal tie sets consisted of four legs. All the legs and tie sets were assumed to equally carry the horizontal joint shear assigned to them. The area of horizontal reinforcement provided for unit B1 was 8% greater than required by the calculations given in Appendix A.3.4.

In using the joint elastic model, certain assumptions need to be made with respect to the horizontal bond force transferred from the beam flexural bars to the concrete arch of the joint core. With reference to Fig. 3.4

$$V_{ch} = \Delta T_c + C_c - V_{col}$$

$$\Delta T_c = \gamma \; (C_s + T)$$

where C_c, C_s and T are the internal concrete and steel forces. The column shear is V_{col}, and ΔT_c is the bond force transferred from the beam bars to the concrete arch. It is assumed that the bond force that is to be transferred will occur in the region of transverse compression supplied by the column axial load and column flexure. In the design, γ was chosen as 0.75 x k, where k is the neutral axis depth factor for the column section at the beam face, calculated according to elastic theory.

The horizontal shear reinforcement used is given in Table 4.3 where it is compared with shear reinforcement used in units B12 and B13.

For unit B2 the calculations indicated that 13.5% more horizontal joint shear reinforcement should have been used. However for this case, i.e. high axial compression on the column, the elastic theory was considered to be conservative. Thus the convenient arrangement of four sets of 6.5 mm bars with 4 legs in each set was used.

As Table 4.3 indicates, where A_h is the total horizontal shear reinforcement between the top and bottom beam reinforcement, considerable reduction in horizontal joint reinforcement is possible when the joint is designed to behave elastically.

Table 4.3 Horizontal Shear Reinforcement

Unit	A_h (mm^2)	Tie diam. (mm)	No. of sets	No. of legs in each set	s (mm)
B1	2027	12.7	4	4	120
B12[1]	4054	12.7	8	4	55
B2	531	6.5	4	4	126
B13[1]	3040	12.7	6	4	75

[1]Tested by Beckingsale[7]

As the spacing, s, was greater, the fabrication of the specimens in the joint region was easier than that of units B12 and B13.

Table 4.4 gives the proportion of horizontal shear force assigned to the concrete shear resisting mechanism using different methods.

Table 4.4 Comparison of the Proportion of Horizontal Joint Shear Force Resisted by the Concrete

	V_{ch}/V_{jh}			
		NZNSEE		
Unit	Elastic Model	Eqn. (2.17)	Eqn. (2.16)	ACI-ASCE 352
B1	0.45	0.54	0.0	0.23
B2	0.79	0.75	0.32	0.23

The calculations for the specimens using the elastic model are contained in Appendix A. The other three methods are described in Chapter two. The two methods on the right-hand side of Table 4.4 apply to inelastic joints. As discussed previously in section 2.5, the ACI-ASCE 352 method suggests the axial load shall be taken as zero for seismic design. The difference between the allowable horizontal joint shear to be carried by the concrete mechanisms, for units B1 and B2 is indicated in Table 4.4, using the ACI-ASCE 352 and NZNSEE methods for inelastic joints. Also shown is the enhanced shear resistance, implied by two methods, as being provided in an elastic joint compared with that implied for inelastic joints.

4.4.2 Vertical Joint Shear Reinforcement

As suggested in Chapter three vertical joint shear reinforcement might be required to complete the truss mechanism. The elastic model suggests that vertical joint reinforcement or vertical axial compression should be capable of sustaining, in addition to loads transmitted to the joint from column flexure, a tensile force of

$$V_{sv} = D_s \sin \beta$$

where D_s is the diagonal component of the truss mechanism, with an inclination of angle β to the horizontal.

A conservative approach appears to be to assign all the vertical tension from the above equation to intermediate bars spaced along the perimeter of the column section in the plane of the applied shear. This would ignore the effect of axial compression on the column, restraining vertically the truss mechanism. However, previous research[15,14] on the behaviour of beam-column joints with low axial load, indicates that without intermediate bars, joint behaviour can be unsatisfactory.

Calculations for vertical reinforcement are also contained in Appendix A where it is assumed the intermediate bars only, act with the column axial load to sustain V_{sv}.

For the case of the high axial load (unit B2), the calculations indicate that no vertical joint shear reinforcement is required. In unit B1 it is found that the intermediate bars are adequate for vertical shear steel.

4.4.3 Confinement

In many situations for seismic design the full benefits of an elastic joint will not be utilized due to the confinement requirements of various codes. These may require more horizontal stirrup-tie reinforcement than that required for shear resistance.

4.5 LOADING SEQUENCE

The loading sequence for the two units is shown in Figs 4.3 and 4.4. For unit B1 twelve load cycles at the elastic limit P_e were applied. A cycle being defined as from zero load to maximum load or under displacement control to maximum displacement, then back to zero. The theoretical elastic limit, as previously described in section 4.2, was determined to be an applied beam shear of 151 kN ($M_y^* = 334$ kNm). Initially it was proposed to load the beams up to 150 kN but when the beam shear applied in the first cycle of unit B1 reached 135 kN it was observed that yield of the outer layer of beam reinforcement had occurred, the highest strains being measured in the region of the outer column bars. Some possible reasons for the inability of the elastic theory to satisfactorily predict the onset of yielding are given below.

The beam bars that are crossed by column bars are influenced by transverse tensile stresses, and strain peaks in this region are not surprising. They have been observed by other researchers.[14] The exactness of using a straight-line strain distribution at the column-beam interface may be questioned because of the possibility of end-effects that may bring about a non-linearity of the strain profile. The modular ratio, n, chosen for the calculations was based on ACI practice[3] but it could have been different.

This observed yielding of beam steel was not considered to be important as the elastic nature of the joint was preserved. The inner layer of bars would certainly not have yielded. The beam tip load which was subsequently applied for the rest of the elastic cycles, in units B1 and B2, was 130 kN.

After the completion of the elastic cycles in unit B1, one cycle in each direction was applied at a displacement ductility factor of 2, followed by two cycles in each direction at a displacement ductility factor of 4. The displacement ductility factor being defined, in these tests, as the ratio of the vertical displacement at the end of the beam, to that occurring from extrapolation of the measured load-deflection curve to the theoretical ultimate load. The axial column load on unit B1 was kept constant at 311 kN ($0.053 f_c' A_g$) throughout the test.

Unit B2 was loaded with twelve reverse cycles at the chosen elastic limit of 130 kN with an applied axial compression of 2890 kN ($0.44 f_c' A_g$). The joint was lightly reinforced and it was felt that

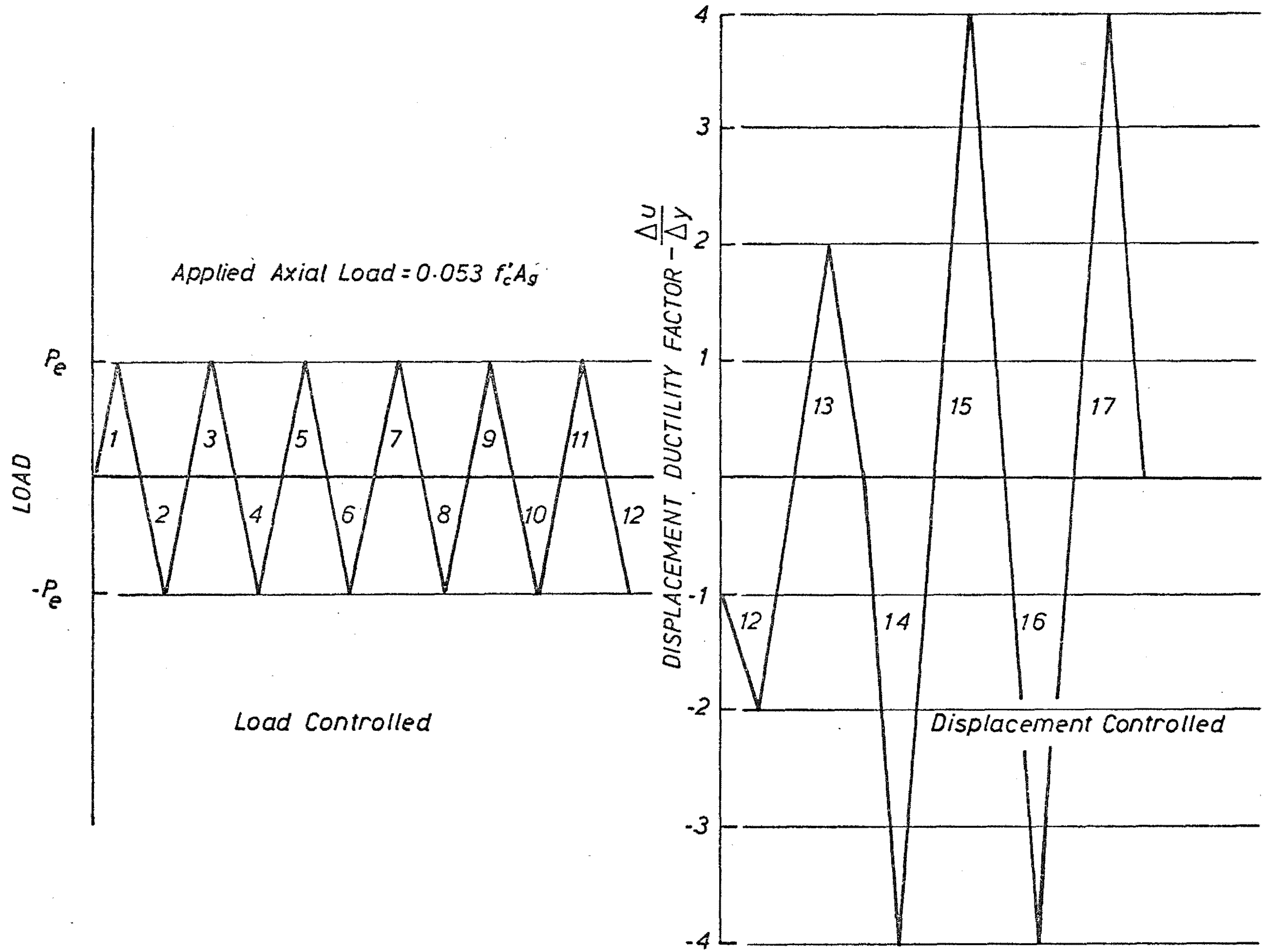


FIG. 4.3 CYCLIC LOADING SEQUENCE FOR UNIT B1

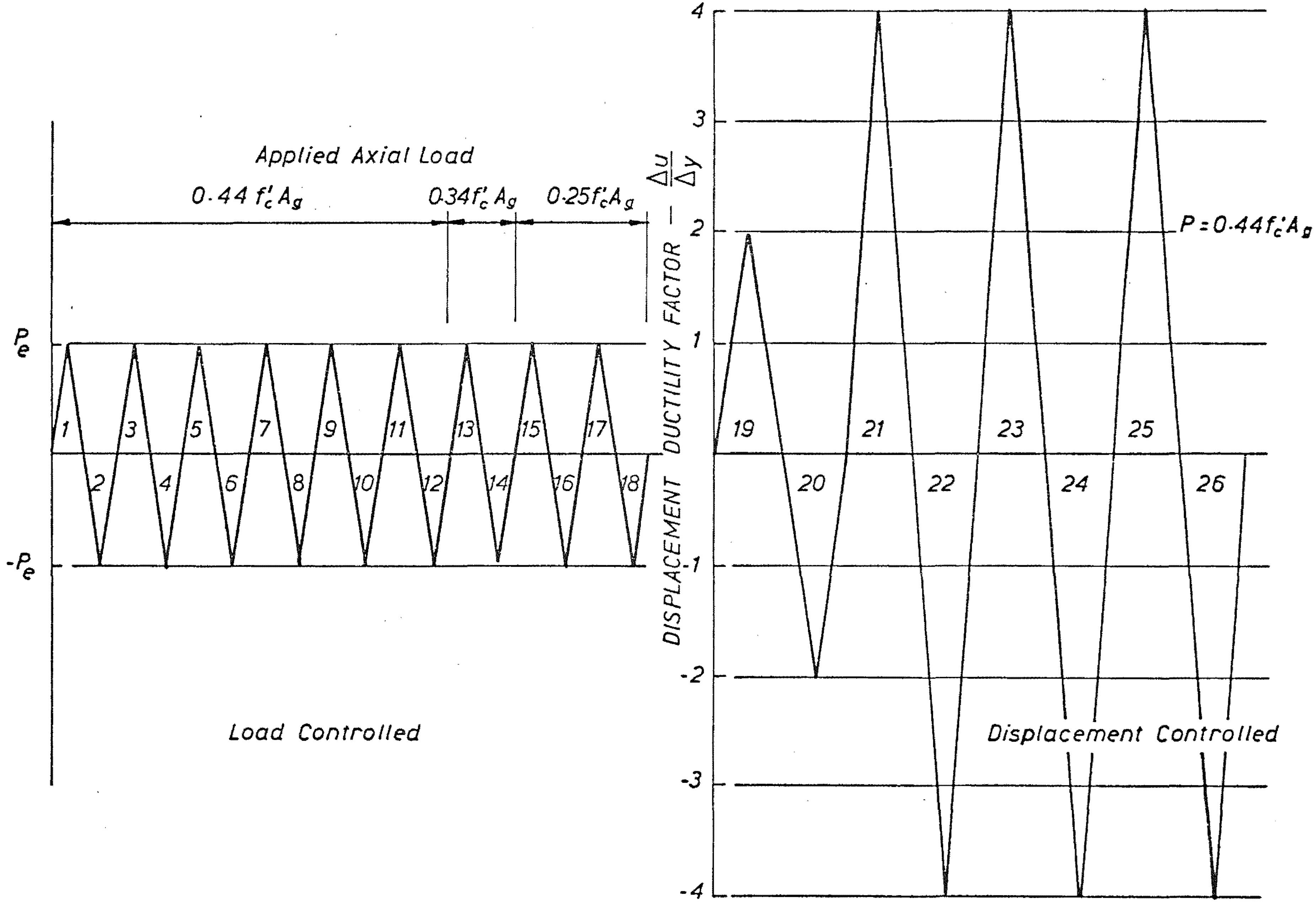


FIG. 4.4 CYCLIC LOADING SEQUENCE FOR UNIT B2

lowering the axial load to 0.25 $f_c' A_g$ might cause yield in the joint ties to occur. Therefore the column load was lowered to only 2240 kN (0.34 $f_c' A_g$). After two cycles at the elastic limit little increased joint deterioration had occurred, with only a marginal increase in joint tie strain. Therefore, four further cycles were applied at the elastic limit with an axial load of 1645 kN (0.25 $f_c' A_g$). With this the initial objectives of the planned test were met. Subsequently the axial load was raised to 2890 kN and inelastic cycles were imposed. Two cycles at displacement ductility factor of two were followed by six cycles at displacement ductility factor of four.

Testing of unit B1 required nine days, while that of unit B2 required eight days. On average, three cycles were completed in a day although the rate of testing was dependent on whether the cycles were in the elastic or inelastic range.

4.6 MATERIAL PROPERTIES

4.6.1 Concrete

The concrete used in both test specimens consisted of 12 mm aggregate. A 75 mm slump and a minimum crushing strength of 28 MPa at 28 days was specified.

Actual concrete crushing strengths were obtained from tests on standard 12" × 6" (304.8 mm × 152.4 mm) cylinders. The concrete cylinders were from the batch concrete placed in the joint core and they were cured along with the specimen. The average of six compression tests, taken at the start of specimen testing, is listed in Table 4.4.

4.6.2 Steel

The beam and column longitudinal reinforcement consisted of deformed bars while all transverse shear reinforcement consisted of plain bars. All steel bars of a particular size were taken from the same batch.

Three samples of each size of bar were tested in tension to determine the yield and ultimate strengths. The average results of these samples are listed in Table 4.4.

Samples of D20 beam flexural bars were also tested in tension to establish the stress-strain curve. This is shown in Fig. 4.6. The length of the yield plateau to the commencement of strain hardening was approximately 13.7 times the strain at first yield. The elastic modulus of the steel used in the D20 bars was found to be 192,000 MPa.

Table 4.4 Material Properties

Bar	Yield strength (MPa)	Ultimate strength (MPa)
D24	427.4	731.0
D20	288.1	448.8
$\frac{1}{2}$" ϕ	345.5	449.7
R 6.5	397.8	514.7
Unit	Concrete Crushing Strength (MPa)	
B1	27.9	
B2	31.5	

Note: All stresses are based on nominal sectional areas.

4.7 LABORATORY PROGRAM

4.7.1 Fabrication of specimens

The units were prepared in a wooden mould, formed from 20 mm plywood which was suitably strengthened so that no movement of the formwork would take place during pouring. The column and beam flexural reinforcement was cut to the required length. The plain bars were guillotined to their correct length. The stirrups were cold bent on a hand operated bending rig using a metal template to form the various stirrup sizes. The joint stirrups had their ends joined by butt welds as shown in Fig. 4.2. Welded stirrups were used, instead of anchorage hooks, to ensure that no slippage of the stirrup would occur. Any such slippage is likely to effect the participation of the stirrups in shear resistance, at this critical region. The column ties, close to the joint, were similarly welded while the rest of the stirrups had hooks for anchorage purposes, formed in accordance with standard ACI practice. The cage was then formed with tie wire and was built to a close tolerance. The built up cage for unit B1 with points for mechanical strain gauges attached is shown in Fig. 4.5.

The units were cast in a horizontal position. The placing of the concrete was continuous with no construction joints. Fig. 4.5 shows unit B1 in the mould ready for concreting. The mould was coated with a thin coating of oil to enable easy stripping of the formwork. The concrete was supplied from a ready mix plant and placed by means of a hopper into the mould. It was vibrated using a 40 mm diameter spud vibrator. The top surface of the units was hand-trowelled and then covered with moist sacking and cured for seven days, before stripping of the mould. Before testing, the surfaces of the units were cleaned and painted white to aid crack detection. The specimens were lifted into the test rig by an overhead crane after 2½ weeks, and testing began approximately 3½ weeks after concreting.

4.7.2 The test rig

The test rig was the same as that used by Beckingsale in his series of tests on beam-column joint units, and a more comprehensive description of the rig can be found in his work.[7] The test rig and specimen are shown in Fig. 4.5. The rig consisted of two parts: a vertical loading frame A, and a lateral loading frame B. The self-equilibrated vertical loading frame was bolted to the strong floor; a pivoting cross arm between the two columns enabled the column axial load to be applied with a 2:1 lever arm ratio. The service load capacity of this system was rated at 3 000 kN. The lateral loading frame consisted of two pairs of 'k' frames attached to the top (1) (see Fig. 4.5) and bottom (2) of the column.

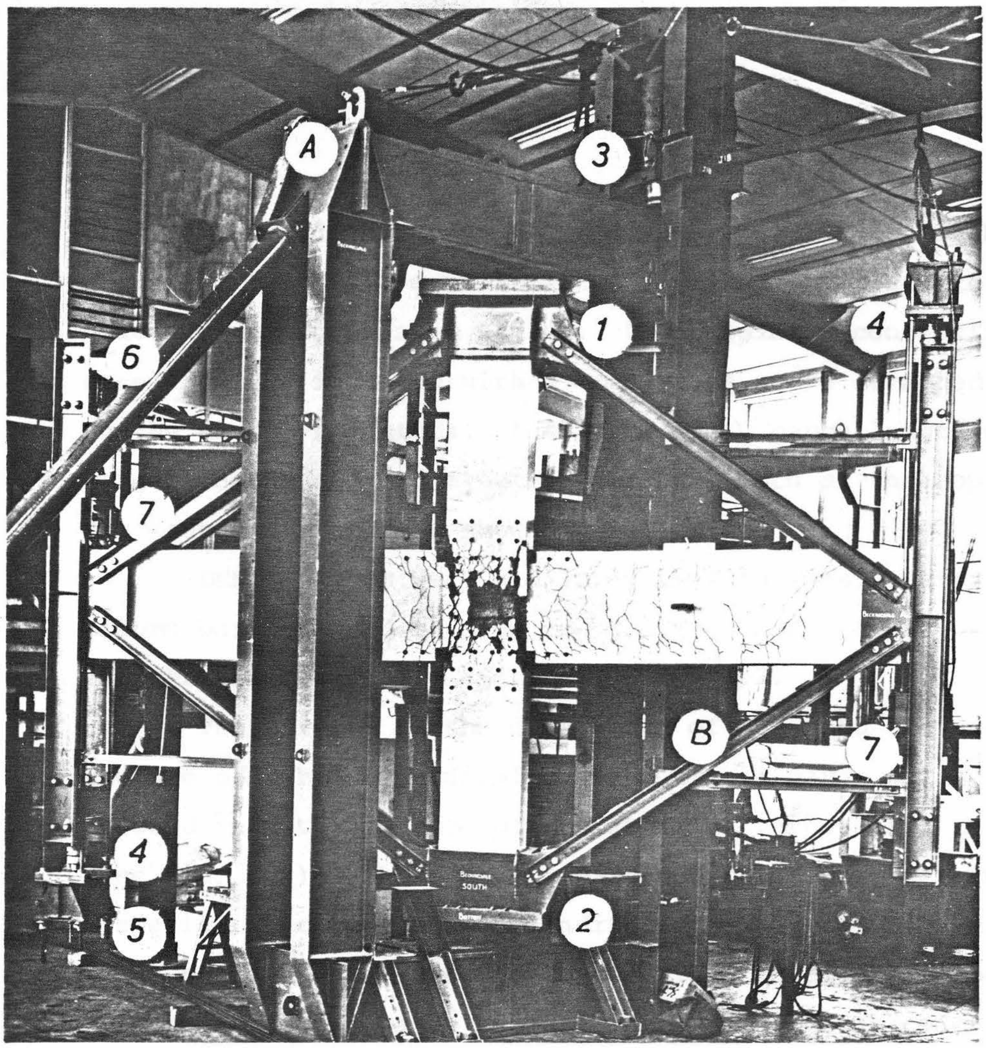
4.7.3 Load application

The column load was applied by means of a 1500 kN capacity hydraulic jack at the top of the portal frame (3). A small adjustment of the load was necessary during the test (see Fig. 4.5).

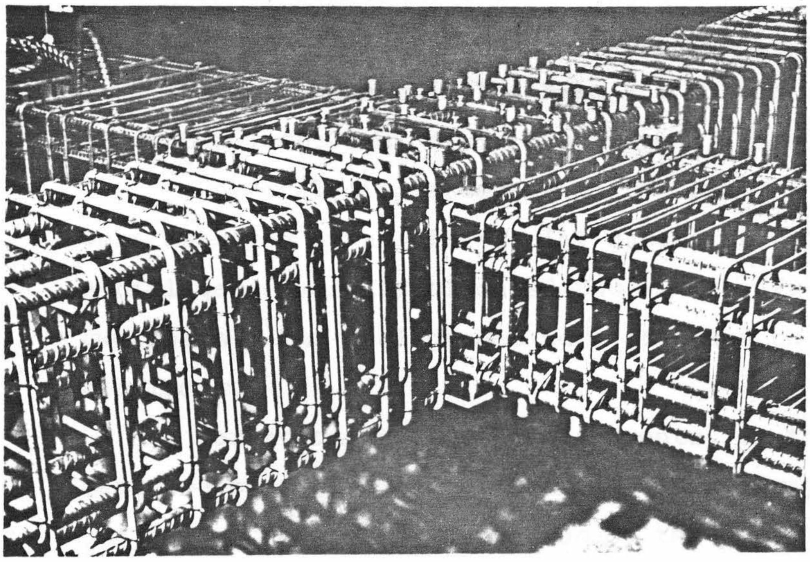
The beam loads were applied by hydraulic jacks, of 200 kN capacity, at (4), reacting through two cross-heads (5), (6) and connected by four high tensile rods. For reversal of load, the jack was shifted to the opposite position (7).

4.7.4 Instrumentation

To measure strains in the longitudinal reinforcement, demountable mechanical (DEMEC) strain gauges were used. The gauges consisted of 10 mm plain bar studs perpendicular to, and welded to, the D20 beam bars



TEST RIG



REINFORCING CAGE

CAGE IN MOULD

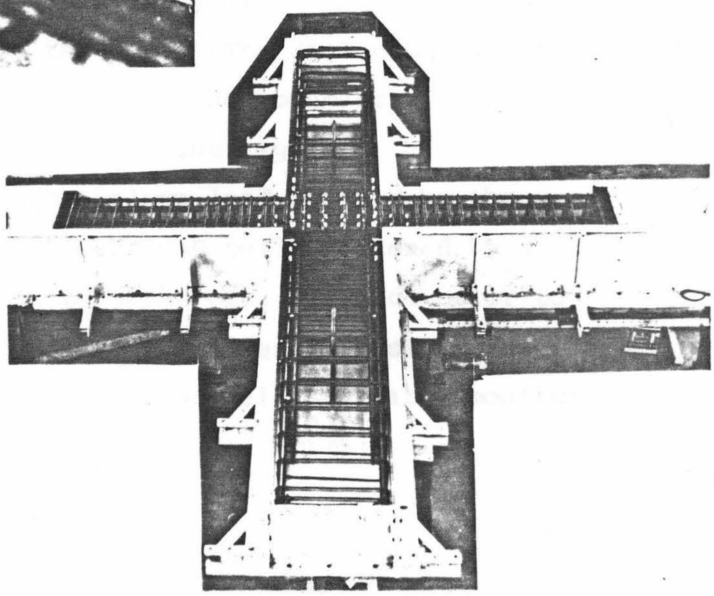


FIG. 4.5

and the D24 column bars. The studs were set flush to the concrete surface. The studs were isolated from the cover concrete by enlarged holes formed by plastic tubing. DEMEC points, consisting of stainless steel buttons indented with a small hole, were attached with sealing wax to the end of the studs. A 4" (101.6 mm) gauge length was used and nine gauge lengths along the outer beam bar, both at the top and bottom of the beam, were used as shown in Fig. 4.2.

DEMEC gauges were similarly used to measure strains in all the column bars passing through the joint on one face only. Nine gauge lengths were used as indicated in Fig. 4.2.

The strains on the outer legs of the main ties were recorded by the use of a BAM mechanical contact strain gauge which uses steel balls pressed into the surface of the steel as targets for the gauge lengths. (see Fig. 4.2). These small, hardened balls were set into the tie by a special punch. A thin cylindrical steel tube was placed around the steel balls and tack welded to the ties. A plastic tube was placed over this steel tube to isolate it from the cover concrete, and subsequently removed after casting, to provide an enlarged hole. The strain gauge was hand held and mechanically operated.

Displacements in the beam and column were measured by dial gauges graduated in 0.01 mm divisions, with a travel of 50 mm.

Vertical displacements in the beam, in the plane of bending, were measured by a pair of dial gauges at the ends of both beams. A dial gauge was also located 840 mm away from the column face on the two beams. The dial gauges were attached to a rigid stand mounted on the floor.

Horizontal column displacements, in the plane of bending, were measured by a dial gauge situated 560 mm away from the top of the beam and 560 mm away from the bottom of the beam. The dial gauge locations are indicated in Fig. 4.1. Measurements of the movement of the test rig were made using a dial gauge at the top of the rig.

An accurate measurement of the column load applied by the hydraulic jack, was made using a 1500 kN load cell monitored by a Budd strain bridge. Calibration of the load cell was made prior to testing, with a 560,000 lb (2,500 kN) Avery Universal testing Machine. The beam loads were measured by a 200 kN load cell similarly monitored by the strain bridge.

A continuous record of the load-displacement characteristics of both beams was produced by two Hewlett-Packard X-Y plotters. This recorded the vertical beam load applied at the end and the vertical end displacement of the beam. The beam load was measured by connecting the plotter in series to the load cell.

Calibration of the plotter for load was achieved by connecting the plotter in series with the Budd strain bridge, the load cell, and the testing machine. LVDT's (Linear Variable Displacement Transducers) driven by a constant voltage power source were used to measure the vertical beam displacements for the plotter. Calibration of the LVDT to the plotter was achieved by using a micrometer with a 20 mm extension.

4.7.5 The testing procedure

At the start of the test the column load was applied by means of a hand pump connected to the hydraulic jack. Three sets of readings of all the dial gauges, DEMEC gauges and BAM contact strain gauges were taken before application of beam loads. Beam loads were applied using hand pumps connected to the hydraulic jacks. The load applied to the beam was monitored by the Budd strain bridge, and three increments were normally applied before the intended maximum load in the cycle, at the elastic limit P_e, was reached. Adjustment of the column load was made after each increment. At each increment, dial gauge readings were taken and on some load cycles a complete set of strain readings was taken for the beam and column bars, and the joint ties. At the maximum of every load cycle the cracks in the joint region, and the surrounding beam and column, were marked by means of an ink pen for easy identification. Photographs of the test specimen were then taken. A full set of dial and strain gauge readings was also taken at this maximum load. The load was then released slowly in several increments, as before, and dial gauge readings were taken. After all the applied beam load had been removed, a full set of dial and strain gauge readings was taken. For load reversal the hydraulic jacks and load cells were shifted using an overhead crane, to their opposite positions, as indicated in section 4.7.3. The load was then applied as before.

After the completion of the test program in the elastic range, beam loads were applied under displacement control. The Hewlett-Packard X-Y plotter was used to monitor the deflection. This enabled the intended approximated displacement ductility factors to be ascertained. Several displacement increments were used before the maximum in the cycle was reached. Adjustments to the dial gauges and LVDT's were necessary when their limit of travel was exceeded. At these displacement increments dial gauge readings, and in the initial displacement ductility cycles, BAM contact strain measurements were made. At the top of each cycle a full set of readings was taken. The load on the beams was then released under load control back to zero load. Towards the end of the test the strains in the ties could not be measured because the large induced strains were beyond the range of the BAM contact strain gauge. Column and beam bar strains were taken at the maximum displacement in some cycles only.

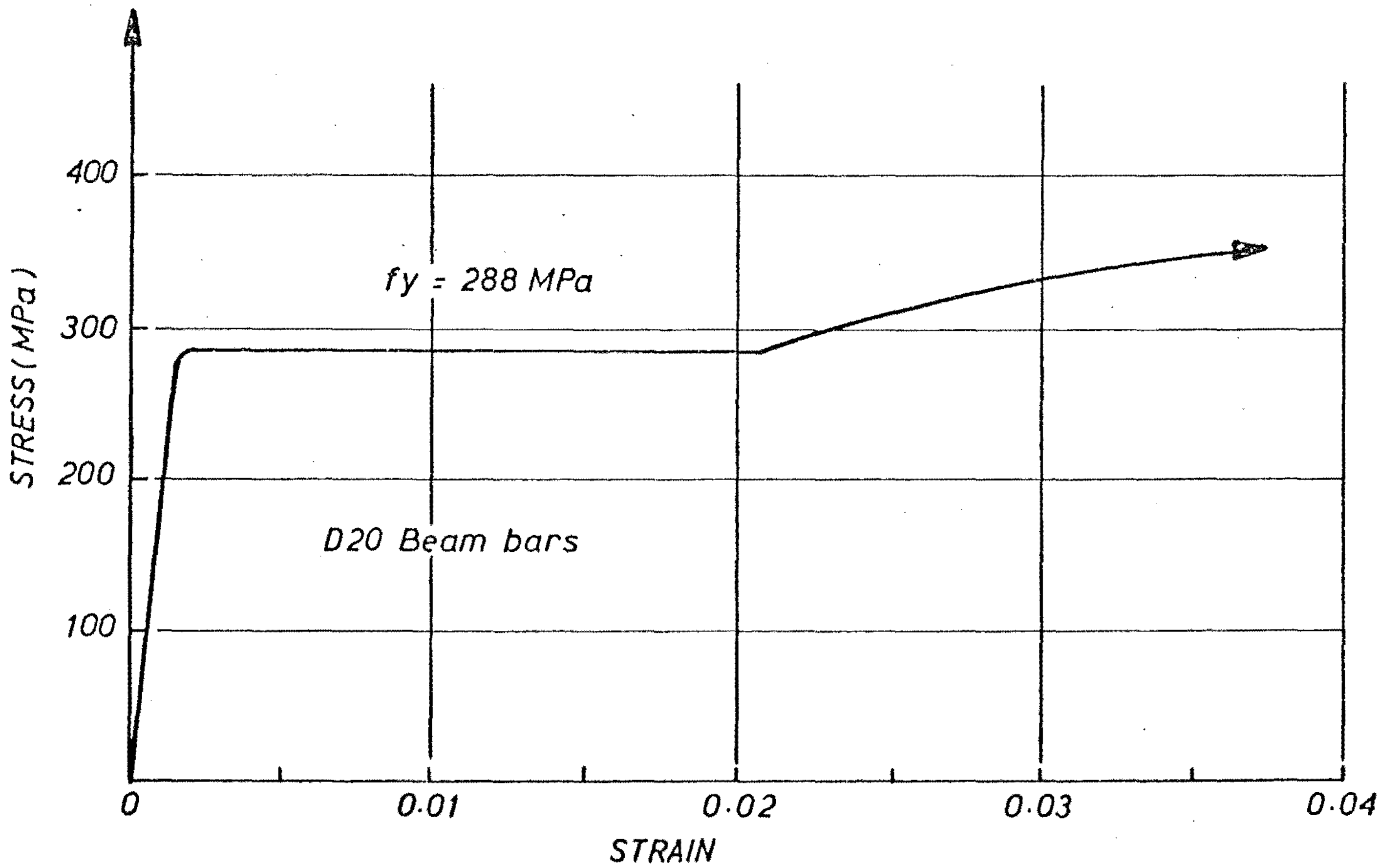


FIG. 4.6 STRESS-STRAIN RELATIONSHIP FOR THE BEAM FLEXURAL REINFORCEMENT

CHAPTER FIVE

TEST RESULTS

5.1 GENERAL BEHAVIOUR

5.1.1 Unit B1

The overall behaviour of the test specimen can be seen in Figs 5.1 and 5.2, which are the load-deflection plots for the two beams of the specimen. The amount of physical deterioration of the joint zone can be ascertained by viewing the accompanying photographs taken at various stages of the test. Cycles 1-11 are the elastic cycles such that the outer layer of beam steel at the end of the cycle is near yield at the column face. The shaded area represents the path of cycles 3(4) - 11(10), which were repeatable in the sense that little stiffness degradation occurred between cycles. Thus for the sake of clarity, all cycles within the elastic limits were not plotted.

The very small shear pinching in the elastic cycles is due to distortions of the panel zone. In the elastic cycles, fine cracks of approximately 0.38 mm maximum width were evenly distributed over a substantial part of the joint panel. The measured anle of inclination of these cracks is in the region of 55^{o}-60^{o} the horizontal (Fig. 5.3). A full length crack running along the diagonal of the joint was noticed when the beam shear was 40 kN ($0.3P_e$) indicating that the tensile splitting strength of the concrete had been exceeded.

In the elastic cycles there was little change along the shear failure plane, and the diagonal cracks closed up after unloading.

After the first inelastic cycle, the average beam load reached an intensity 4% greater than the theoretical ultimate load for the beam. With two cycles at a displacement ductility of 4, the beam load had dropped to 70% of the theoretical ultimate, accompanied by drastic degradation in stiffness. The diagonal cracking became more extensive and dislodged particles, and yielding in the ties prevented closure of these cracks after unloading. The cracks in the inelastic cycles were approximately of 1.2 mm maximum width. As the photographs show, the cover concrete had been lost and the core concrete had deteriorated to

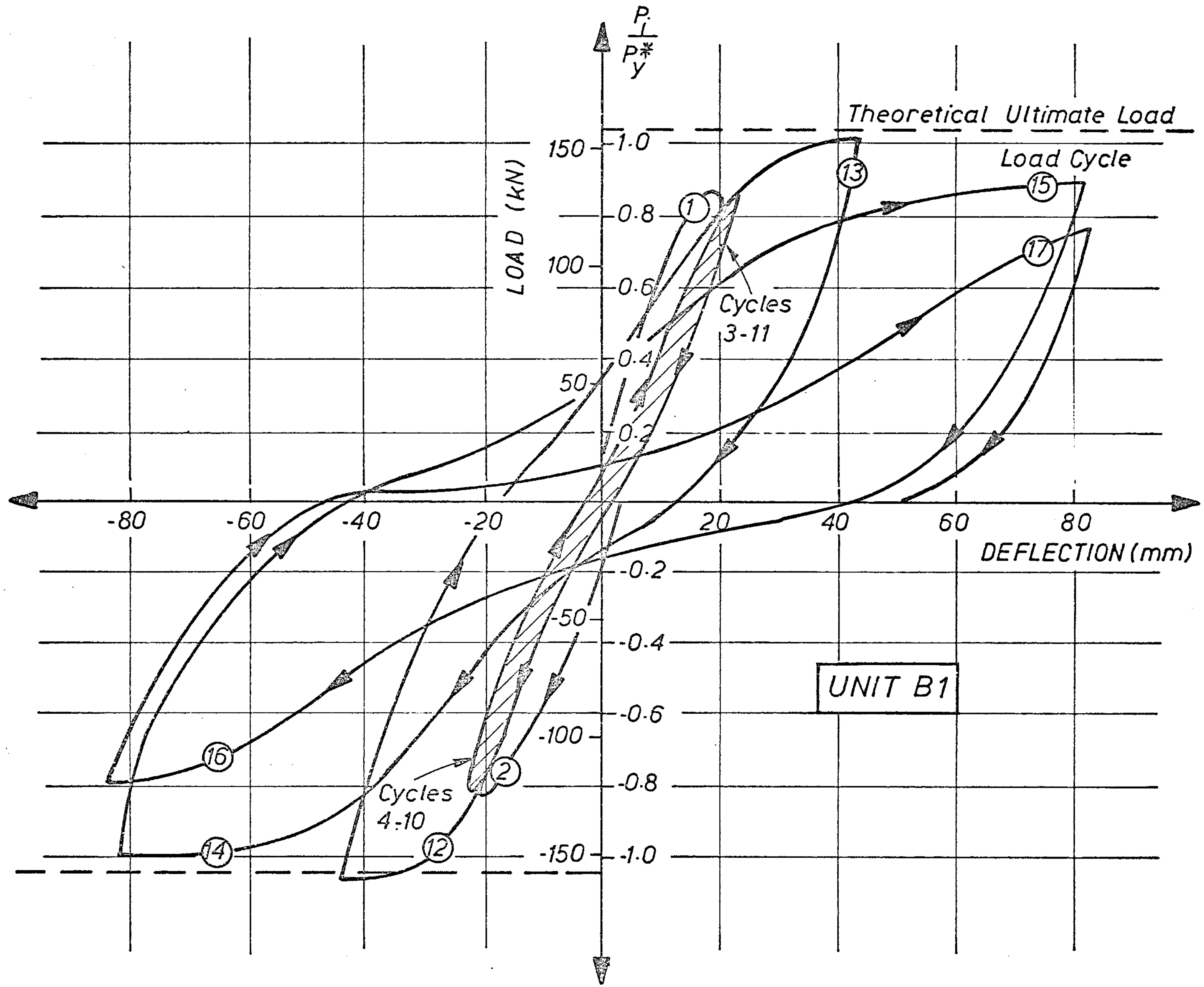


FIG. 5.1 LOAD-DEFLECTION RELATIONSHIP FOR WEST BEAM - UNIT B1

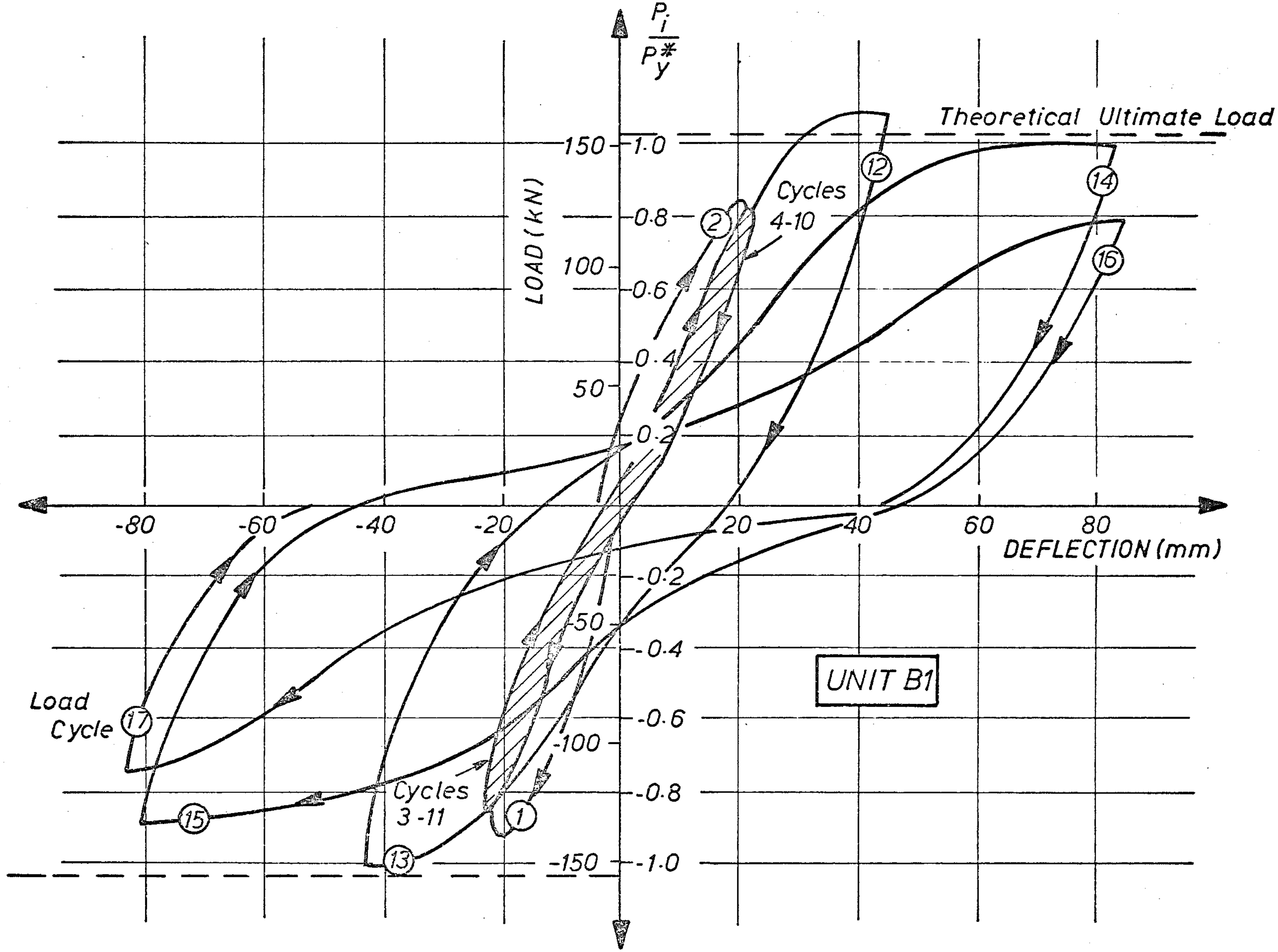


FIG. 5.2 LOAD-DEFLECTION RELATIONSHIP FOR EAST BEAM - UNIT B1

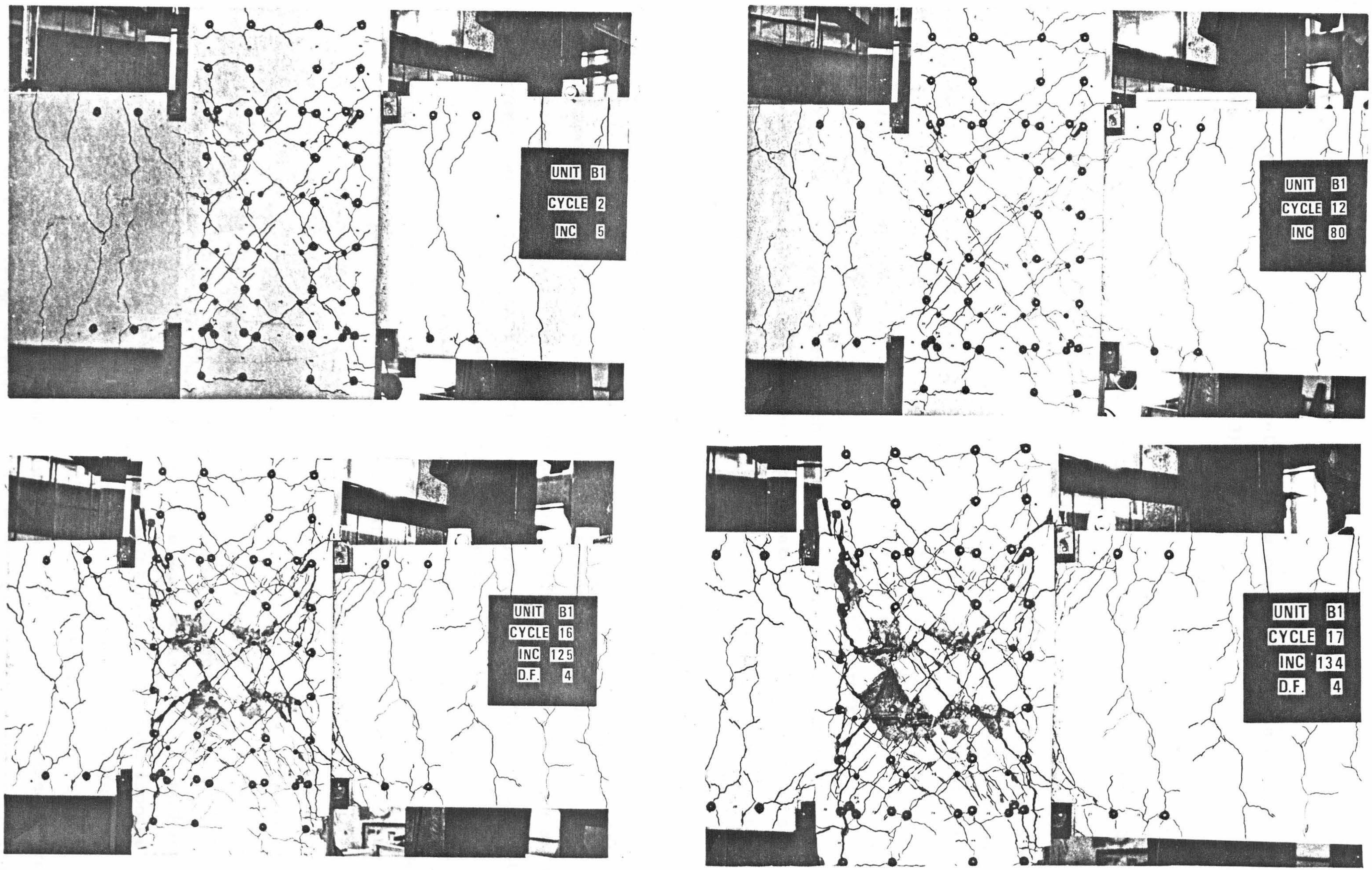


FIG. 5.3 THE JOINT REGION OF UNIT B1 AT VARIOUS STAGES OF TESTING

the extent that it was completely lacking cohesion and was easily broken up by hand (see Fig. 5.3).

5.1.2 Unit B2

Load-deflection plots for the east and west beams of unit B2 are given in Figs 5.4 and 5.5. The elastic cycles 1(2) - 11(12) are represented by the shaded curve and show little stiffness degradation. There was little shear distortion in the panel zone when the high axial column load was present. With the lowering of the axial compression load from 0.43 P_o to 0.24 P_o there was an increase in beam deflection, i.e. loss of stiffness, shown in cycles 17 and 18, due to greater distortions in the panel zone. The photograph shown in Fig. 5.6 indicates that, in the initial elastic cycles, the cracks in the core developed along the length of the intermediate column bars. These small cracks, inclined at approximately 70^o to the horizontal, appeared to be initiated by the intermediate bars which weakened the tension field.

Maximum crack widths in the elastic cycles were approximately 0.33 mm. With the subsequent lowering of the axial load and the application of further elastic load cycles, the cracking in the joint became more extensive and, although not easily detectable, the crack inclination appeared to have changed (Fig. 5.6). Due to the high axial compression load, concrete strut action appeared to be the predominant mode of shear resistance throughout the test.

In the first inelastic cycle, the load attained was 5% greater than the theoretical ultimate load for the beam, but upon load reversal significant load reduction occurred. After three cycles at ductility of 4, in each direction, the beam load had dropped to 52% of the theoretical ultimate. The cracking was more extensive, maximum crack widths of approximately 1.2 mm were measured. After several inelastic cycles, large diagonal cracks had opened up with yielding of the joint ties. These cracks covered the full width of the joint core (Fig. 5.6) and caused spalling of the cover concrete surrounding the column bars, nearest to the beams. Yielding of the joint ties would have imposed transverse tensile strains on the core concrete and this severely weakened the compression field. As Fig. 5.6 shows, spalling of the cover concrete occurred, accompanied by a disintegration of the concrete core. At this stage of the test the column was losing compression load

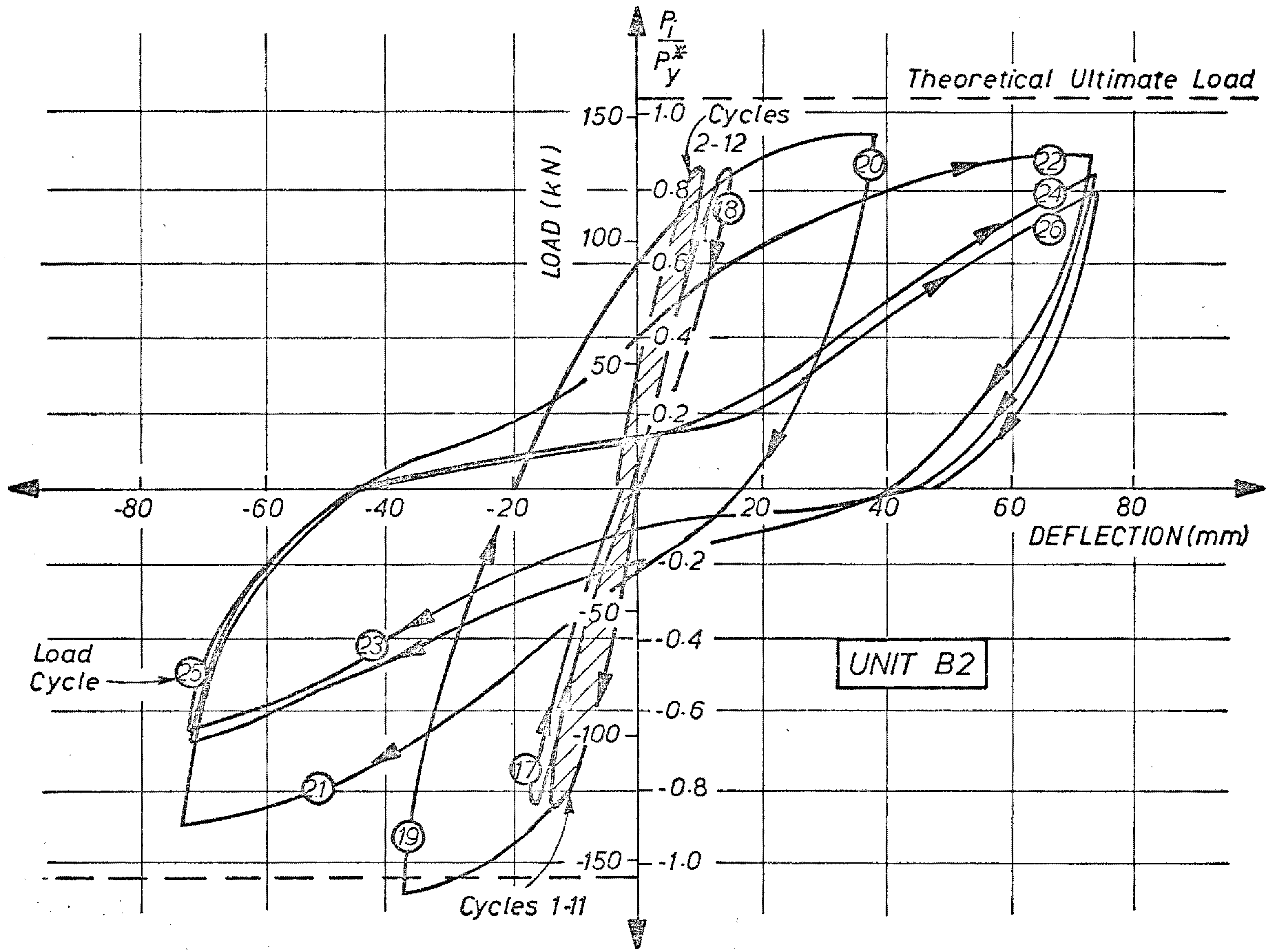


FIG. 5.4 LOAD-DEFLECTION RELATIONSHIP FOR EAST BEAM - UNIT B2

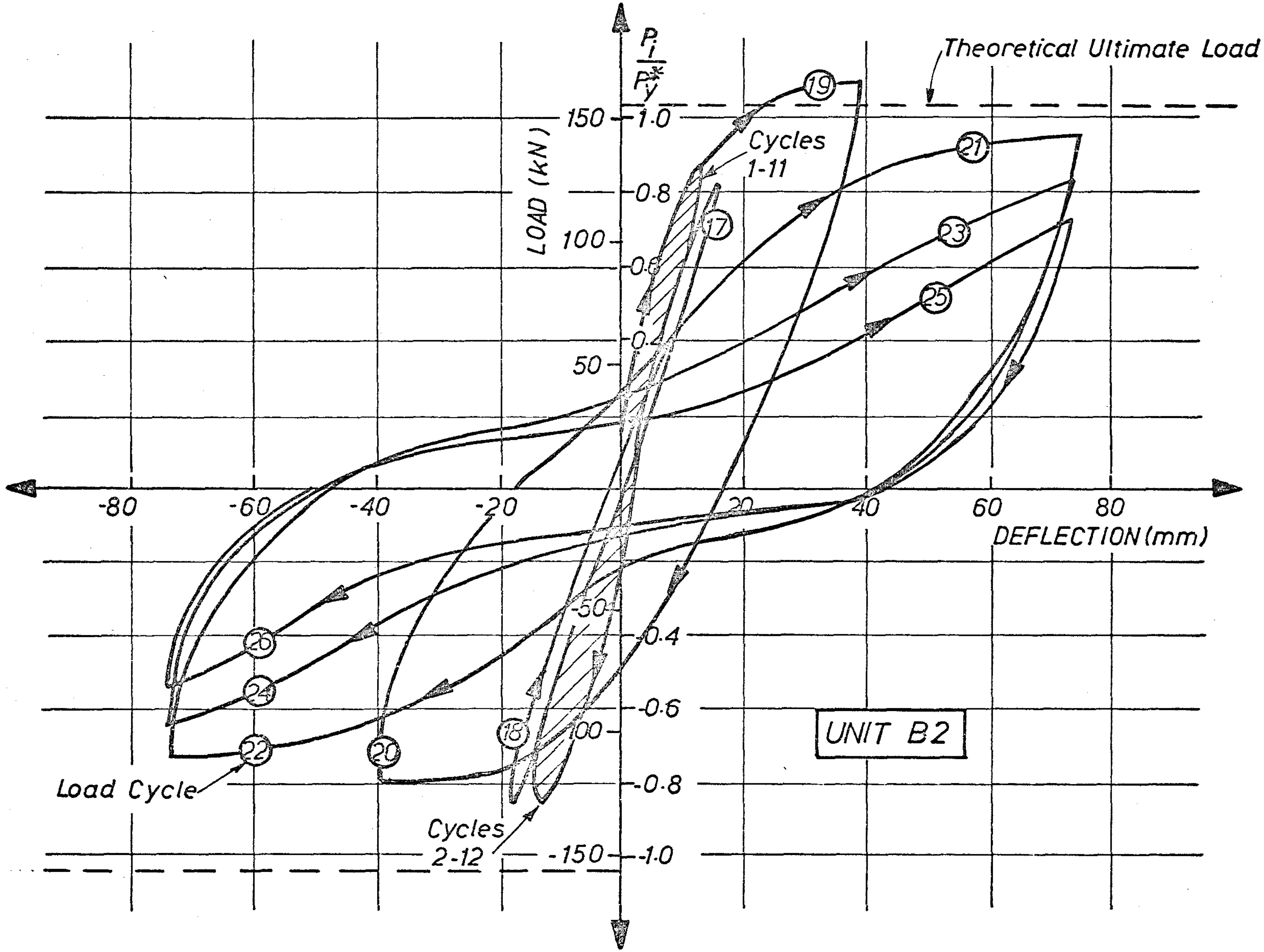
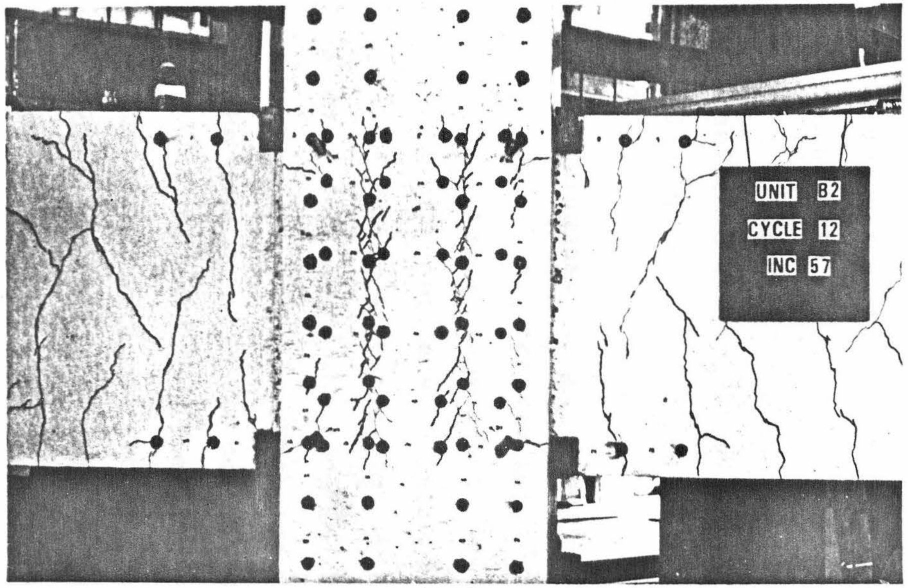
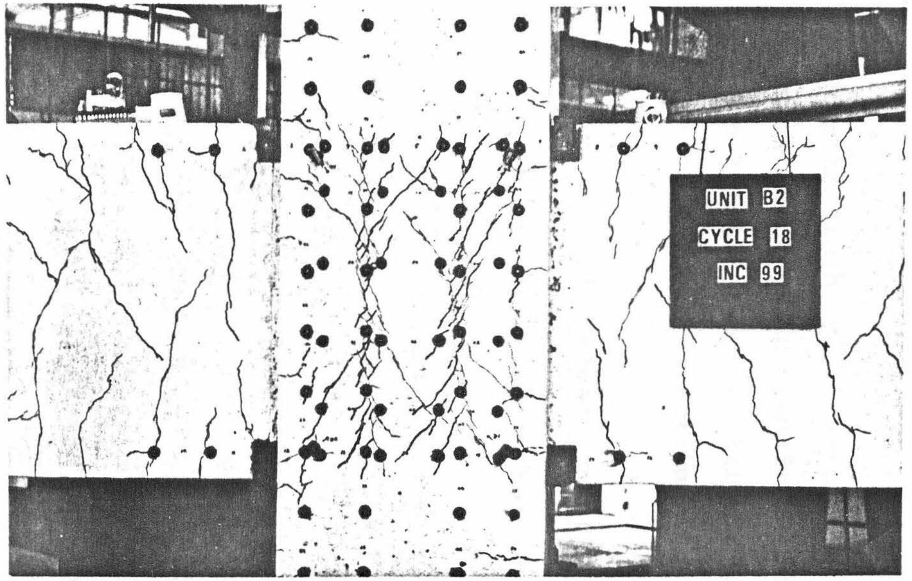


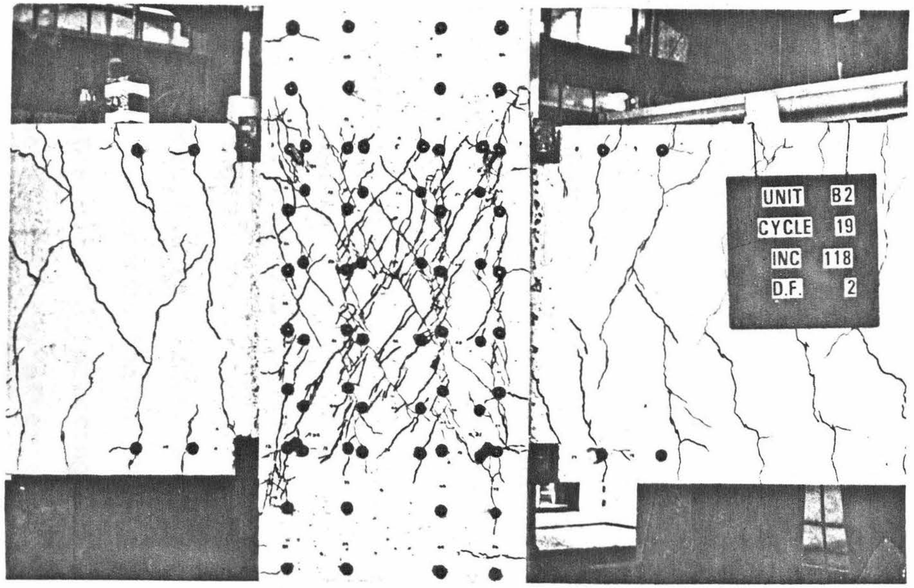
FIG. 5.5 LOAD-DEFLECTION RELATIONSHIP FOR WEST BEAM - UNIT B2



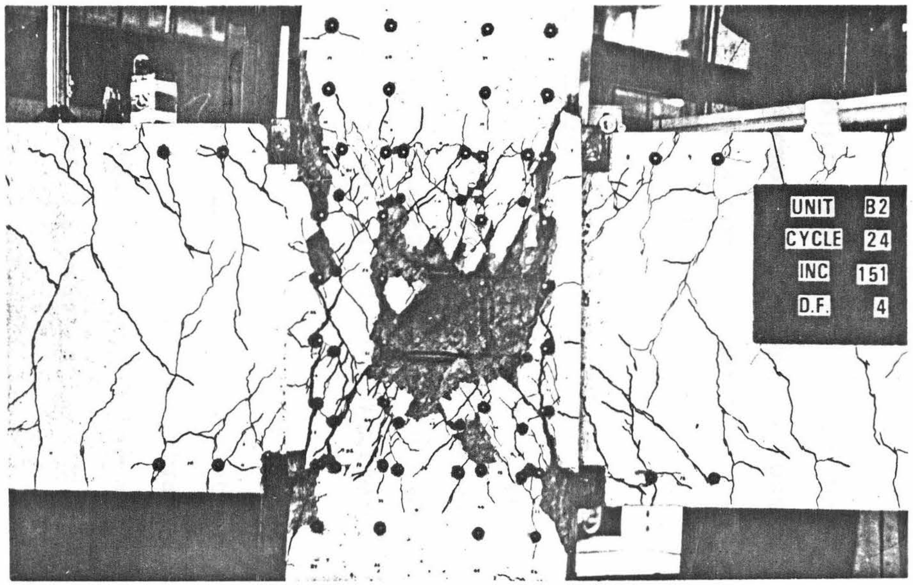
$0.44f'_cA_g$



$0.25f'_cA_g$



$0.44f'_cA_g$



$0.44f'_cA_g$

FIG. 5.6 THE JOINT REGION OF UNIT B2 AT VARIOUS STAGES OF TESTING due to crushing of the concrete in the joint core. The column bars were required to transfer a much greater compression force across the joint. Eventual failure appeared to be a compression failure of the diagonal concrete strut.

5.1.3 Comparison of Units B1 and B2

The load-deflection behaviour of the two units showed that the beam deflection for unit B2 in the elastic load cycles was only 55% of that occurring in unit B1. This indicates, along with absence of shear pinching, that the high axial load resulted in a stiffer joint core.

In the elastic cycles a full corner to corner diagonal crack appeared in unit B1, whereas in unit B2 the cracking was concentrated near the intermediate column bars. This indicated the predominance of the axial compression in unit B2. With the lowering of the axial load in unit B2, cracks developed further and extended across the joint.

The inelastic behaviour of the two units was comparable with respect to crack patterns. The larger cracks that formed towards the end of the test ended diagonally across the joint in both units.

From the load-deflection plots it can be seen that in the inelastic cycles the strength degradation on load reversal was slightly greater in unit B2 than in unit B1.

The two specimens appeared to have different modes of failure. The diagonal cracks in unit B1 appeared to have been caused mainly by the shear stresses introduced into the joint core. The diagonal cracks became wide after yield of the joint stirrups. This was similar to a diagonal tension failure. In unit B2 the cracking appeared to be mainly due to the high axial compression causing diagonal splitting. The yielding of the ties in the inelastic cycles weakened the compression field. This led to the eventual compression failure of the core concrete as described in section 5.1.2.

Neither test specimen was able to meet the requirements of NZS 4203[16] with respect to ductility. The recommendation of this code of practice is that primary members of a seismic resisting system should not lose more than 30% of their strength, after eight reversals at a displacement ductility factor which corresponds with a displacement ductility of 4 for the entire structural system. The corresponding ductility for a subassembly such as the one studied would need to be at least four but in certain frames it could be more. Unit B1 had lost 30% of its strength after only four reversals at ductility of 4, while unit B2 had lost 48% of its strength after six reversals at ductility of 4.

5.2 BEAM FLEXURAL REINFORCEMENT

5.2.1 Unit B1

Plots of strain distribution for the outer layers of bars through the joint are shown in Figs 5.7 and 5.8. The plots were obtained from the average of the strain readings taken on both sides of the beam. Unintended yield of the beam bars occurred at the column face. The likely reasons for this are discussed in Chapter four. The peaks in strain at locations 7 and 12 are due to high residual tensile strains remaining after the occurrence of yield in cycle 1. Compression strains on load reversal in the elastic range were not large enough to induce net compression strains in the bars at these locations.

The strain distribution along the bars through the joint is approximately linear in the elastic cycles except at locations 7 and 12 on account of yielding in the first cycle.

The strain distribution in the inelastic cycles is given in Fig. 5.8. It is evident that there is progressive penetration of yield into the joint core with the higher imposed ductilities. These inelastic strains were converted to stresses using a Bauschinger analysis computer programme, developed by Spurr[17] from theoretical equations based on Ramberg Osgood functions.

Fig. 5.9 shows stress distributions along beam bars through the joint at yield, ductilities of 2 and 4, based on this stress-strain analysis. It can be seen that the stress distributions through the joint are approximately linear. The dashed line indicates the theoretical stress distribution at the applied load, P_e, derived from the elastic analysis. As described in Chapter four, there are several reasons why the elastic theory might not satisfactorily predict the actual observed stress values. The larger deviations from theory occur in the region of

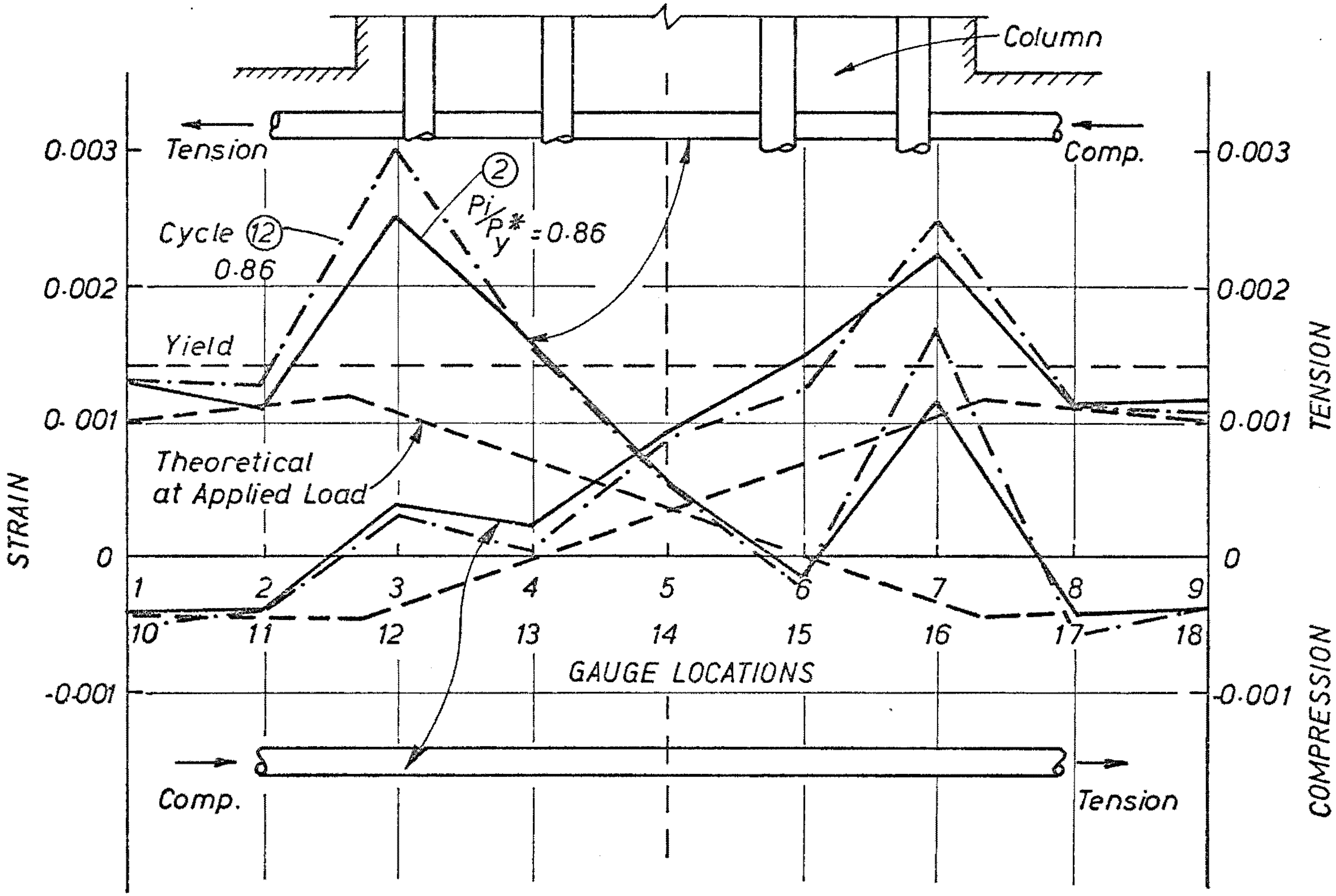


FIG. 5.7 BEAM BAR STRAINS IN THE JOINT CORE OF UNIT B1

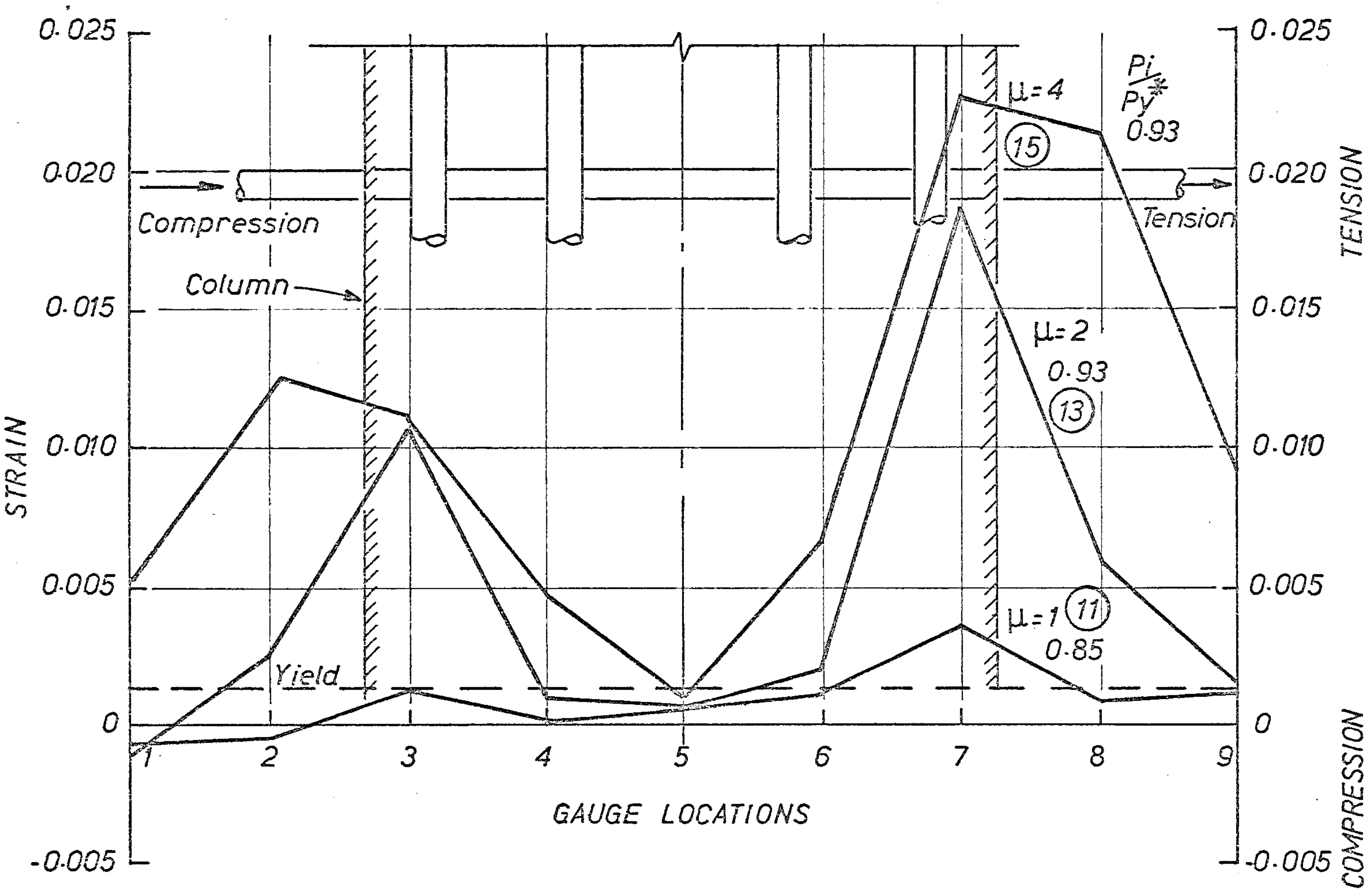


FIG. 5.8 BEAM BAR STRAINS IN THE JOINT CORE OF UNIT B1

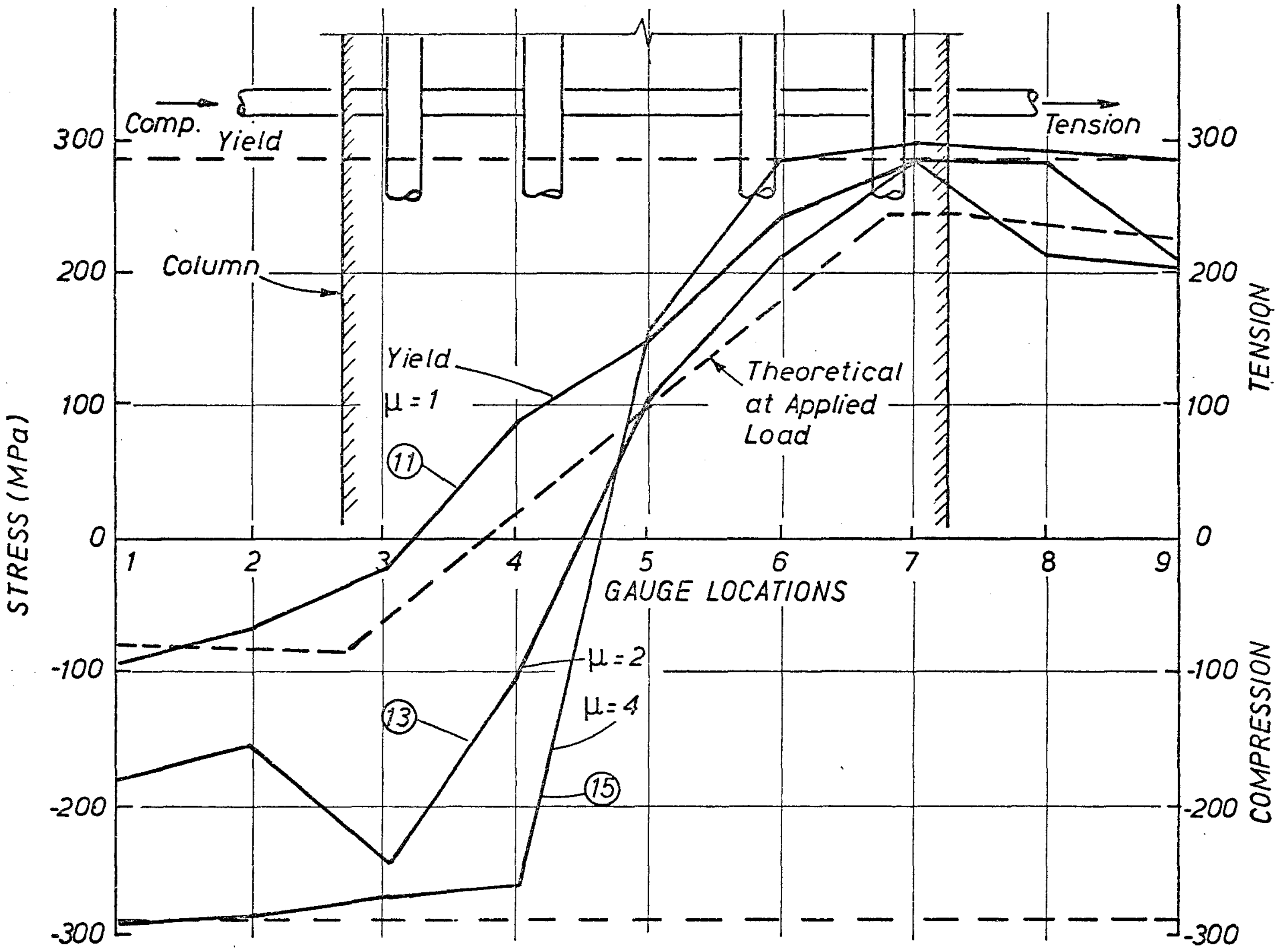


FIG. 5.9 STRESSES ALONG THE TOP BEAM BARS WITHIN THE JOINT OF UNIT B1

the column faces. Although the tensile stresses are higher and the compression stresses are lower at these positions, the rate of change of the bar stress through the joint is about the same as the theory would predict. This rate of change (slope) of bar stress is proportional to the bond stress existing in the bar.

A comparison of average bond stress calculated from an elastic analysis of beam forces and that from the experimental results is presented in Appendix B.

The ACI code[3] requirements imply that a development length of 354 mm would be required for D20 bars to sustain the tensile and compression forces calculated from an elastic analysis.

The associated average bond stress is 4.0 MPa. From the observed stress distribution for the top load of the elastic cycle plotted in Fig. 5.9, it is evident that the maximum tension stress in the beam bar occurred near the column bar closest to the column face. The maximum compression stress in the same beam bar occurred outside the joint core (gauge location 1). The calculated average bond stress between the points of observed maximum tension and the compression bar stress at the column face on the opposite side is 3.8 MPa.

With the inelastic cycles there was some penetration of yield strain into the joint, from the tension face of the column. After the third cycle, at displacement ductility of 4, the bar had yielded also in compression. This indicated that the moment of resistance of the beam at the column face would have been provided by the steel couple alone. The efficient transfer of horizontal shear forces across the joint by the diagonal concrete strut had been severely diminished, i.e. the concrete compression force at the column face was very small. At cycle 15 with a displacement ductility factor of 4, an average bond stress of 9.4 MPa was indicated in the central region of the joint core.

5.2.2 Unit B2

The strain distributions for the outer layer of beam bars of unit B2 are plotted in Figs 5.10 and 5.11. The strain distribution in the elastic cycles was linear, and as the applied beam tip loads were never above 130 kN, in contrast to what occurred in unit B1, the strain peaks near the column face due to unintended early yielding were largely avoided.

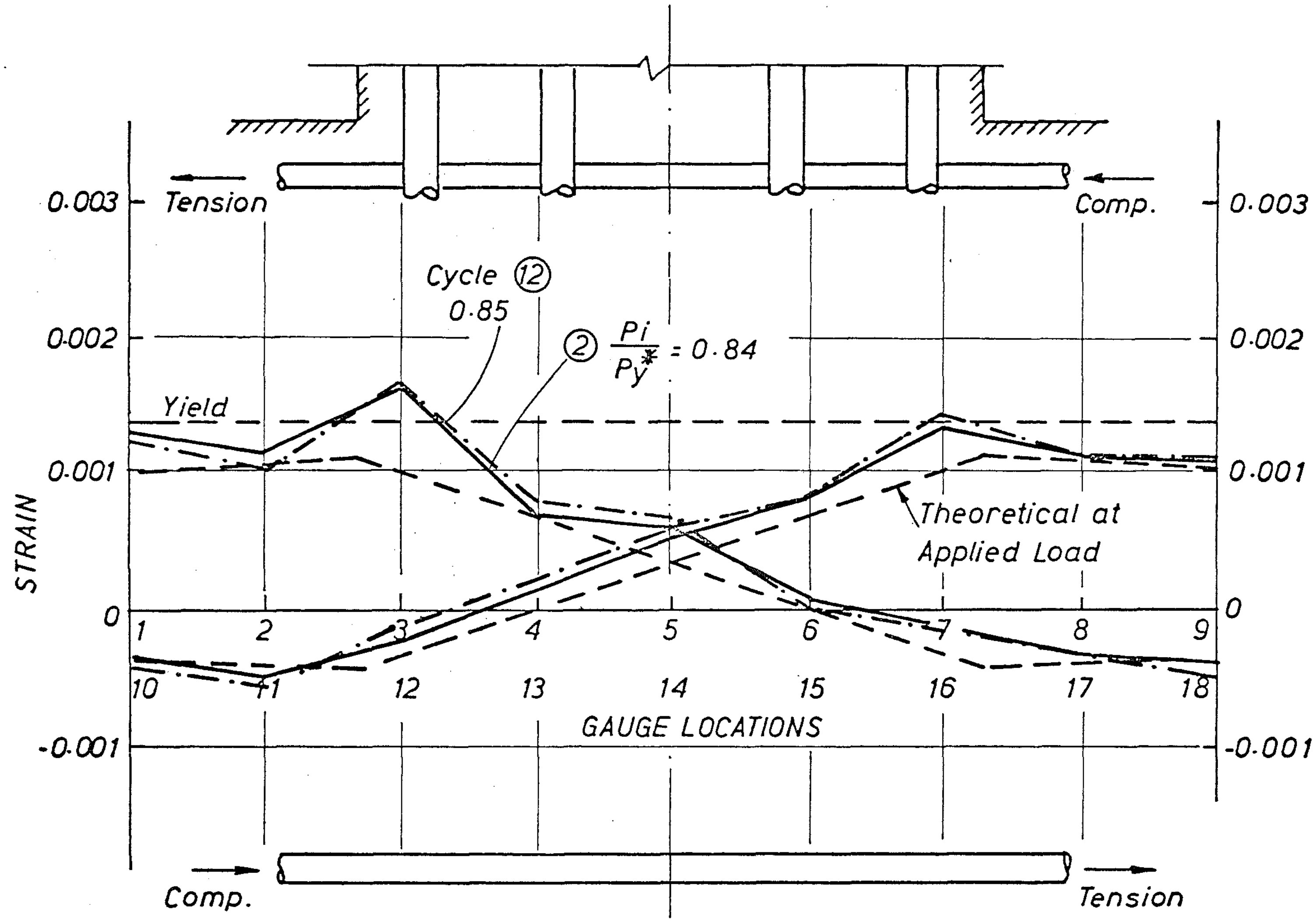


FIG. 5.10 BEAM BAR STRAINS IN THE JOINT CORE OF UNIT B2

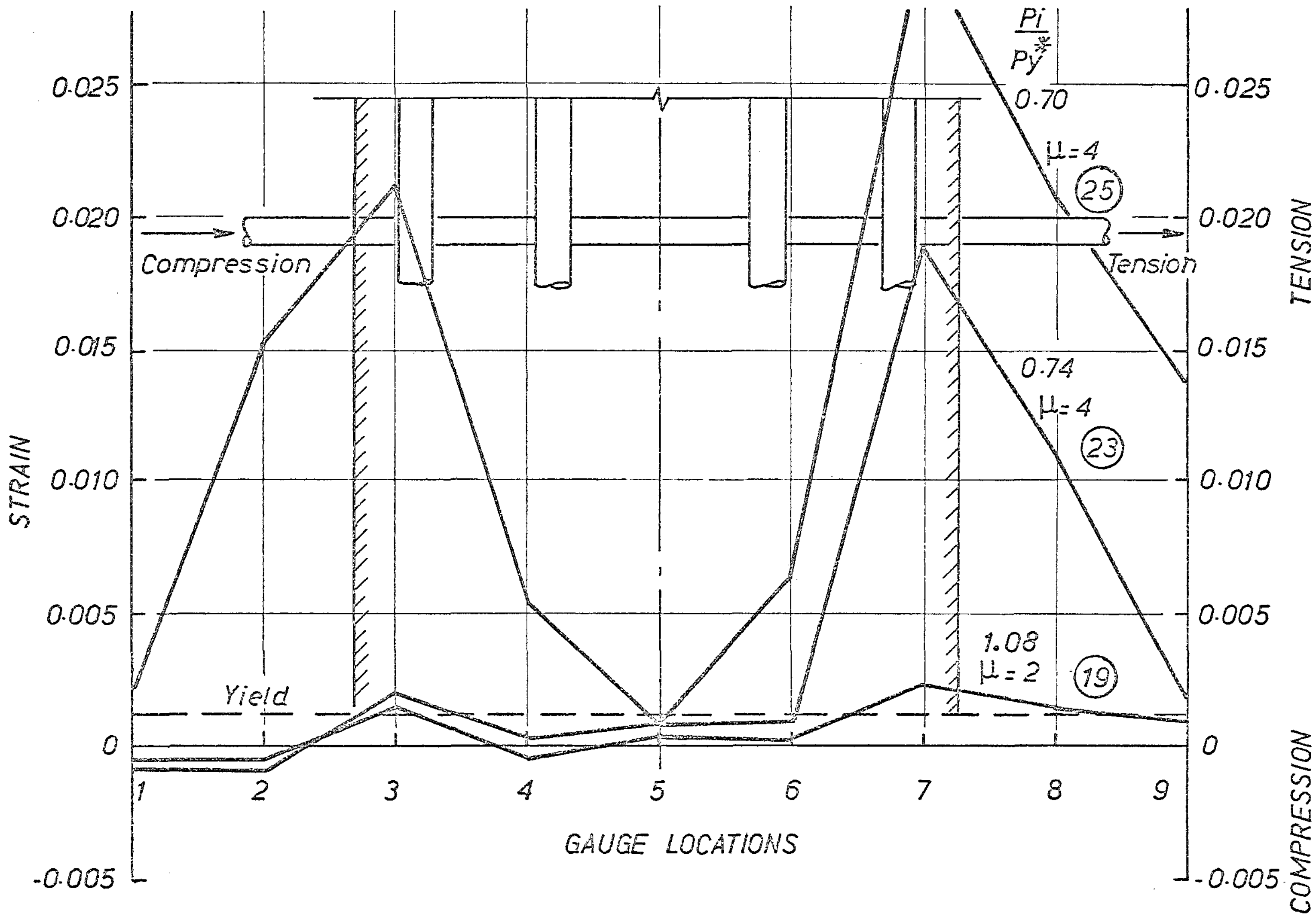


FIG. 5.11 BEAM BAR STRAINS WITHIN THE JOINT CORE OF UNIT B2

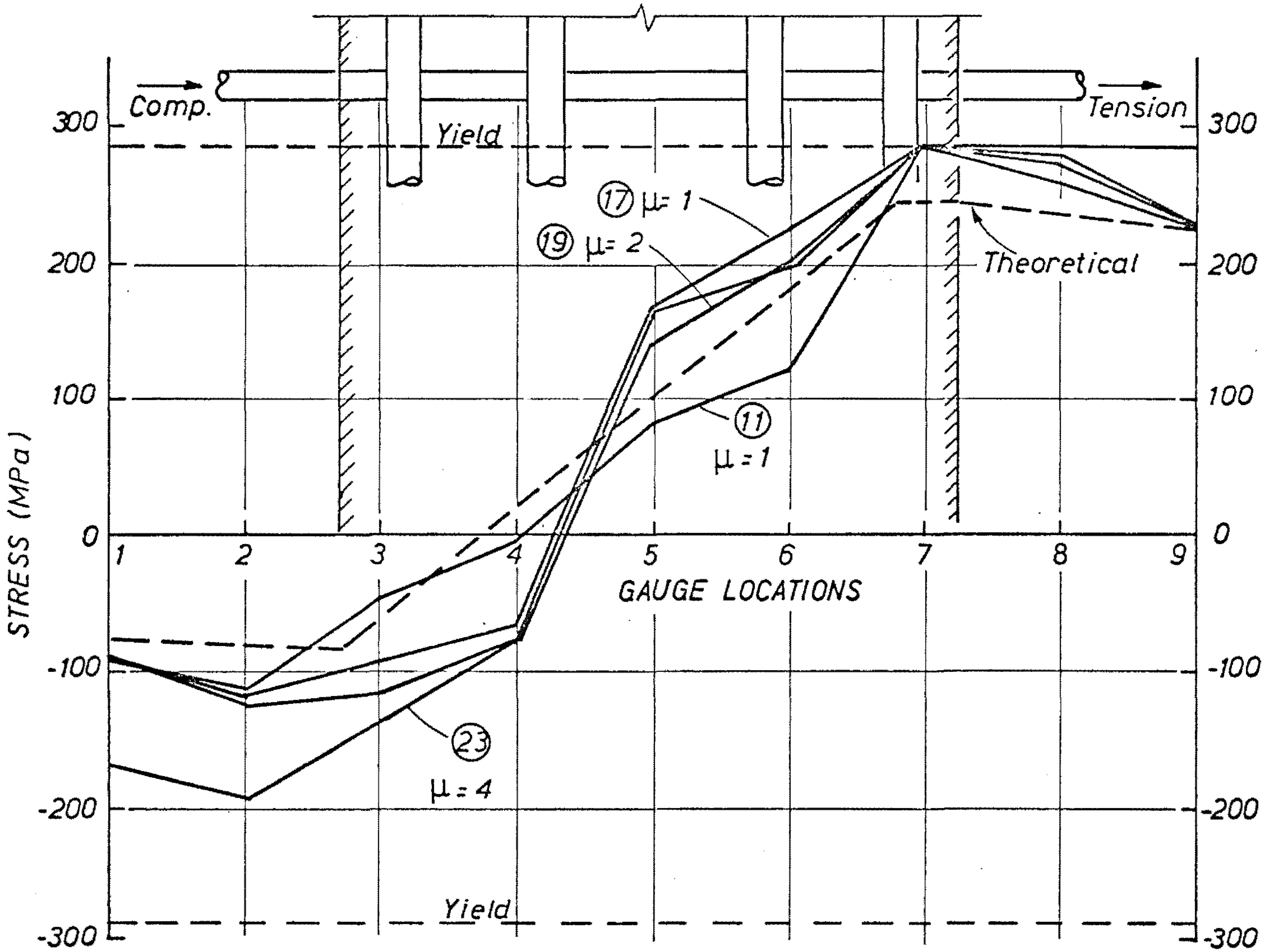


FIG. 5.12 STRESSES ALONG THE TOP BEAM BARS WITHIN THE JOINT OF UNIT B2

Fig. 5.11 shows the strain distribution for the top beam bar in the inelastic load cycles. Penetration of yield into the joint with the higher ductilities can be observed.

The stress distributions, derived from the Bauschinger analysis of Spurr,[17] are plotted in Fig. 5.12. The stress distribution at the top load of the elastic cycle closely matches the theoretical stress distribution. The calculated average bond stress, between the point of observed maximum tension and the compression bar stress at the column face on the opposite side, is 4.1 MPa.

5.2.3 Comparison of Unit B1 and Unit B2

The strain distribution for unit B2 was more linear than in unit B1, due to the absence of yielding as described in sections 5.2.1 and 5.2.2. The theoretical stress distribution also more closely matched the stress distribution observed in unit B2 than that in unit B1. For comparable stages of loading, the penetration of yield along the beam bar, from the compression and tension faces of the column, was not as significant in unit B2 as in unit B1. In unit B2, after three cycles at ductility of 4, the average bond stress was 54% of the average bond stress in unit B1 after only two cycles at ductility of 4. This indicates that the large axial compression, present in unit B2, enabled a more favourable transfer of bond forces than that in unit B1.

However, in the inelastic cycles there was a slightly larger degradation of load in unit B2 than in unit B1. The failure of the joint in unit B2 does not appear to be attributable to the breakdown of bond.

5.3 COLUMN REINFORCEMENT

5.3.1 Unit B1

The longitudinal strain patterns for the column bars on the south face are shown in Figs 5.13, 5.14, 5.15 and 5.16. The theoretical strain pattern is plotted for the observed elastic limit of the beam, P_e. Not unexpectedly there is considerable non-linearity of strain distribution along the column bars through the joint. The strain peaks which are more evident in Fig. 5.16 are due to the column bars carrying tensile stresses across the diagonal cracks in the joint core.

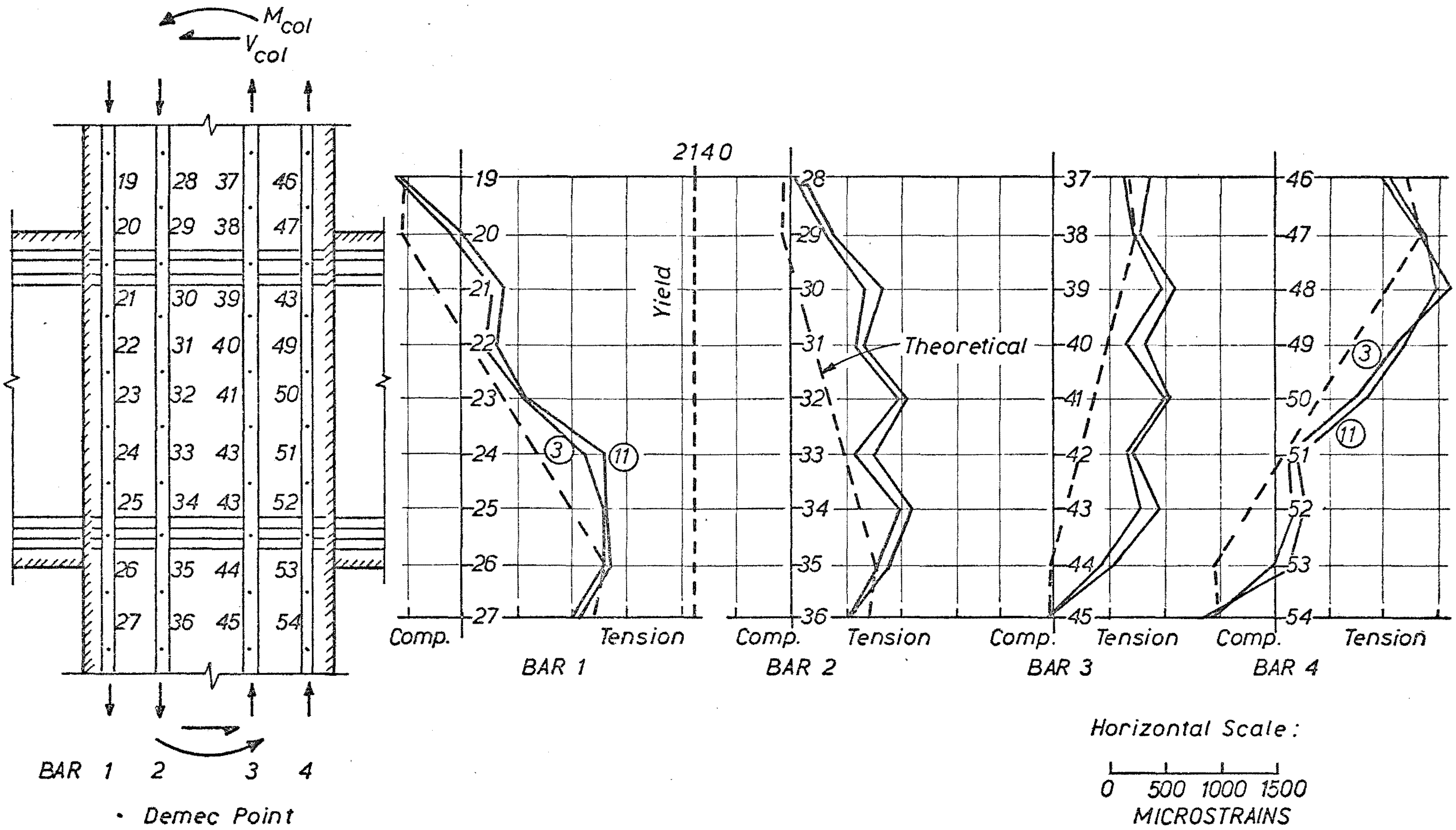


FIG. 5.13 COLUMN BAR STRAINS IN THE JOINT CORE OF UNIT B1

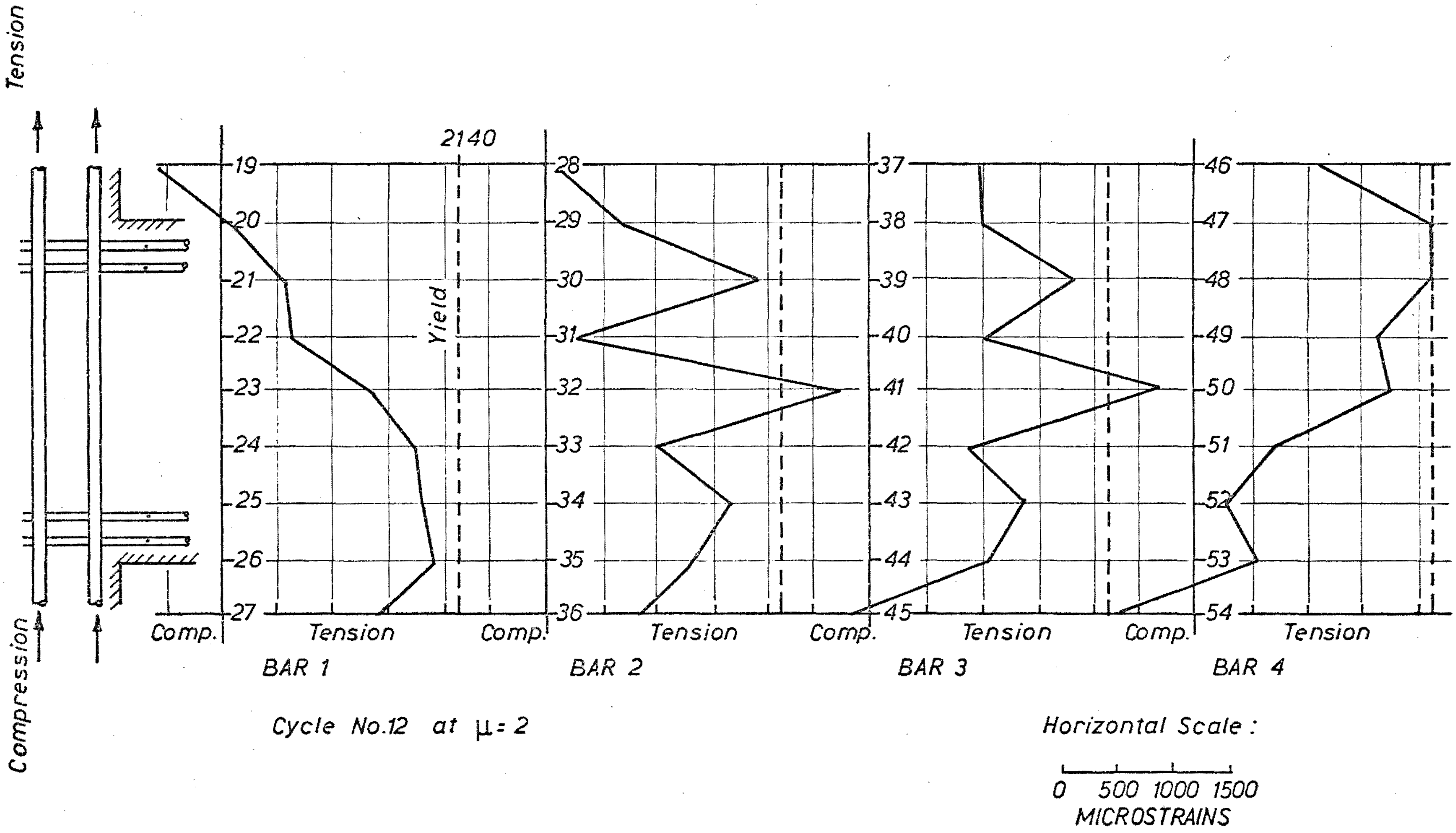


FIG. 5.14 COLUMN BAR STRAINS IN THE JOINT CORE OF UNIT B1

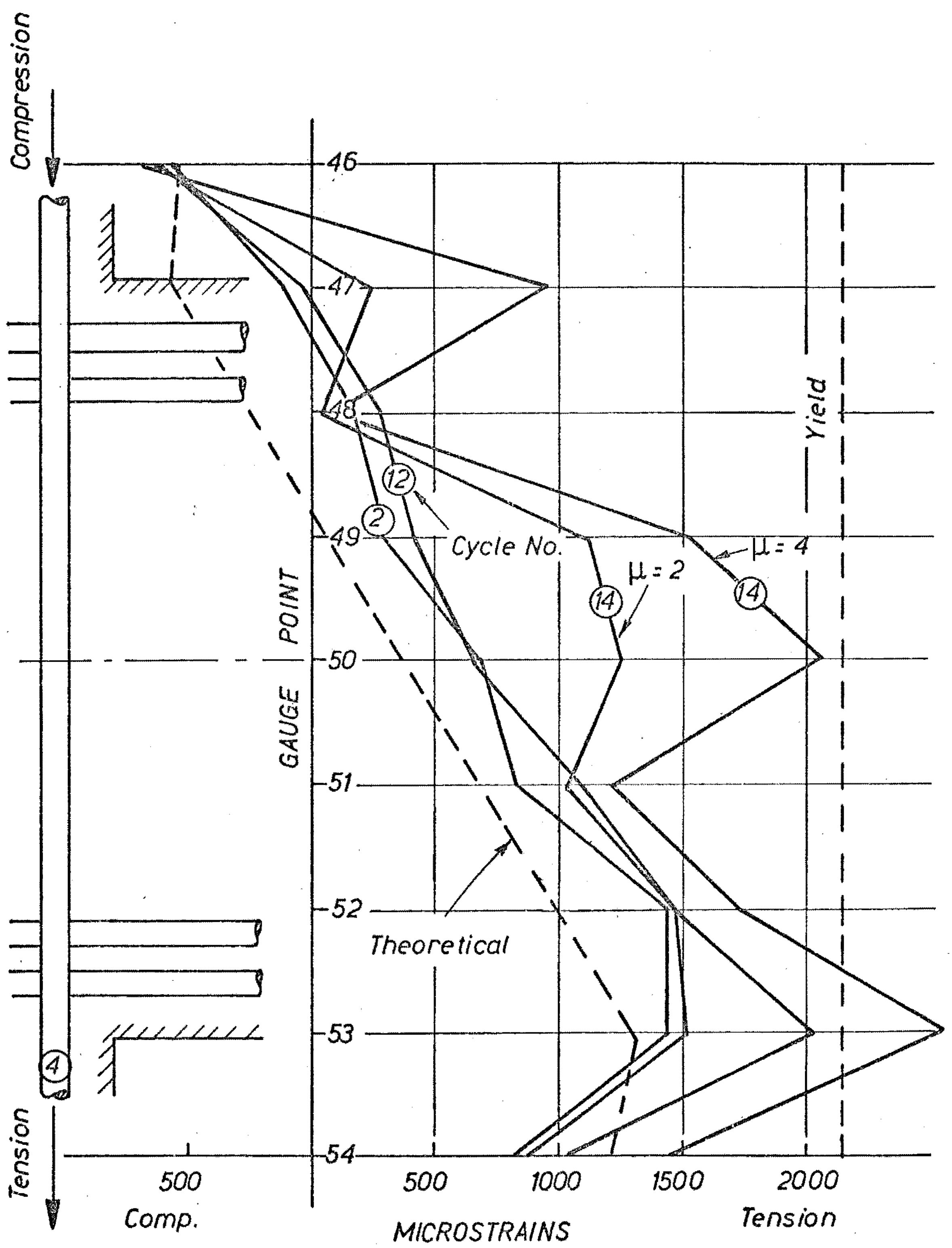


FIG. 5.15 STRAINS ALONG COLUMN BAR 4 OF UNIT B1

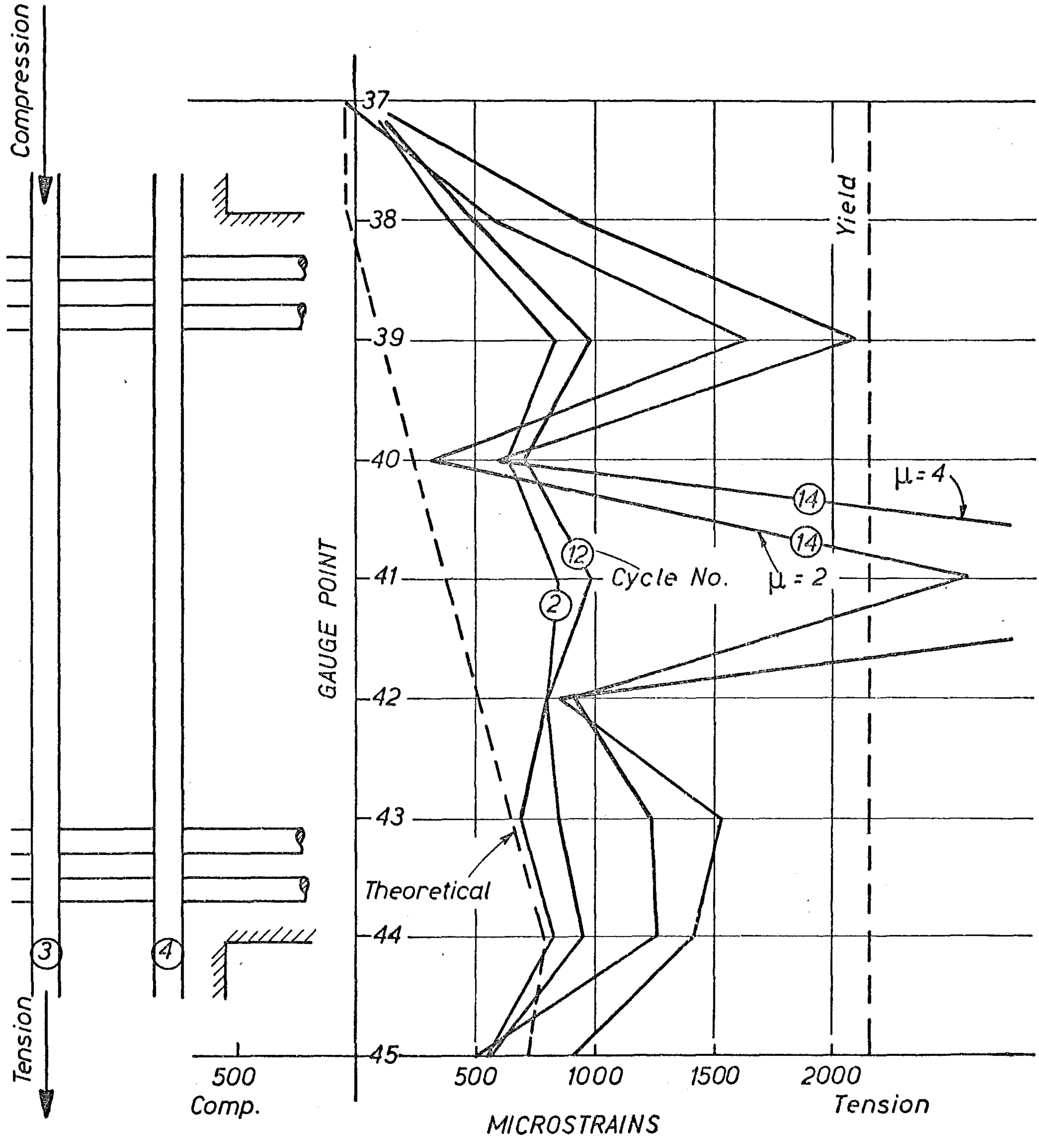


FIG. 5.16 STRAINS ALONG COLUMN BAR 3 OF UNIT B1

In Fig. 5.13 the strain distribution is more linear than that of Fig. 5.14 because at this stage the joint is still responding within its elastic limit and the crack widths remained relatively small (0.4 mm), tending to close up after unloading. After several inelastic cycles the diagonal cracks became larger (1.2 mm).

The strain profiles indicate that the column bars carry tensile strains, in addition to those from flexure, as part of the joint truss mechanism made up of horizontal and vertical shear reinforcement. The intermediate bars appear to participate to a greater extent in this truss action than those near the beams. Longitudinal splitting cracks in the concrete occurred parallel to these perimeter bars. Figs 5.15 and 5.16 show that the column bars had yielded in tension at certain locations after several inelastic cycles. However, at this stage, wide diagonal cracks have crossed some strain gauges along the colum, and the dowel displacement imposed on the base may have seriously effected the strain readings.

5.3.2 Unit B2

The longitudinal strain distributions for the column bars through the joint are plotted in Figs 5.17, 5.18, 5.19 and 5.20. The compression strains in the intermediate bars (Fig. 5.17) are in some cases higher than the compression strains in the outer bars for the same respective gauge locations. This is most probably due to the larger compression stresses in the centre of the joint core caused by joint shear. This is consistent with Fig. 3.1. The cracks which formed in both directions (Fig. 5.6), as described in section 5.1, are sufficient to weaken the compression carrying capacity of the concrete. After inelastic cycles were applied, the column bar strain distributions became more non-linear as the cracking was more extensive. As Fig. 5.20 shows, the columm bars yielded in compression towards the end of the test. Photographs taken at ductility of 4 (Fig. 5.6) indicate the extent of damage to the joint core. With the severe weakening of the compression carrying capacity of the concrete, the column bars were required to sustain large compression strains in addition to that due to flexure of the column. At this stage the inadequacy of the ties and breakdown of the concrete led to buckling of the column bars. This buckling occurred in the plane of the frame and, although it was hard to detect, it would seem to have been most severe in the corners of the

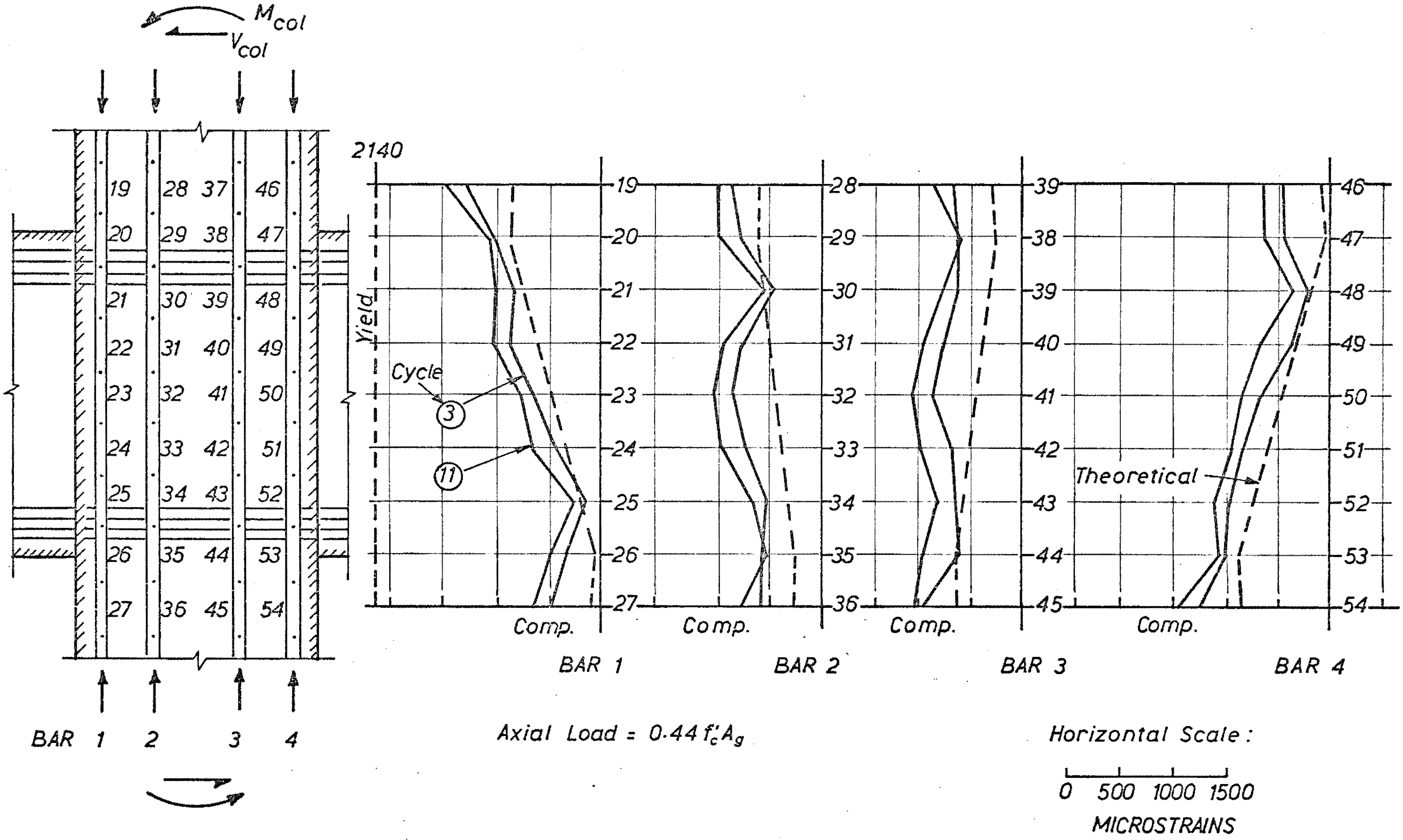


FIG. 5.17 COLUMN BAR STRAINS IN THE JOINT CORE OF UNIT B2

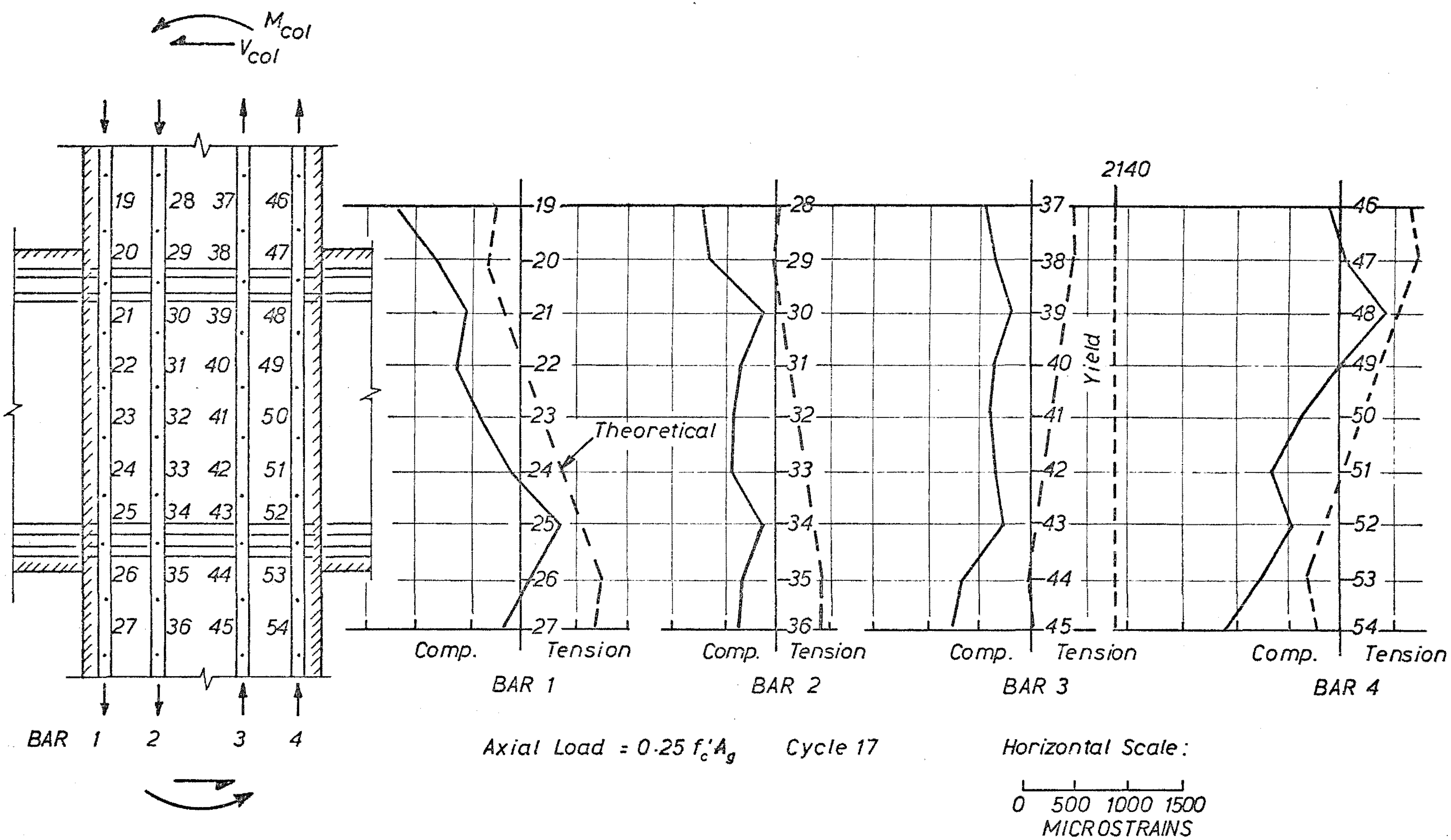


FIG. 5.18 COLUMN BAR STRAINS IN JOINT CORE OF UNIT B2

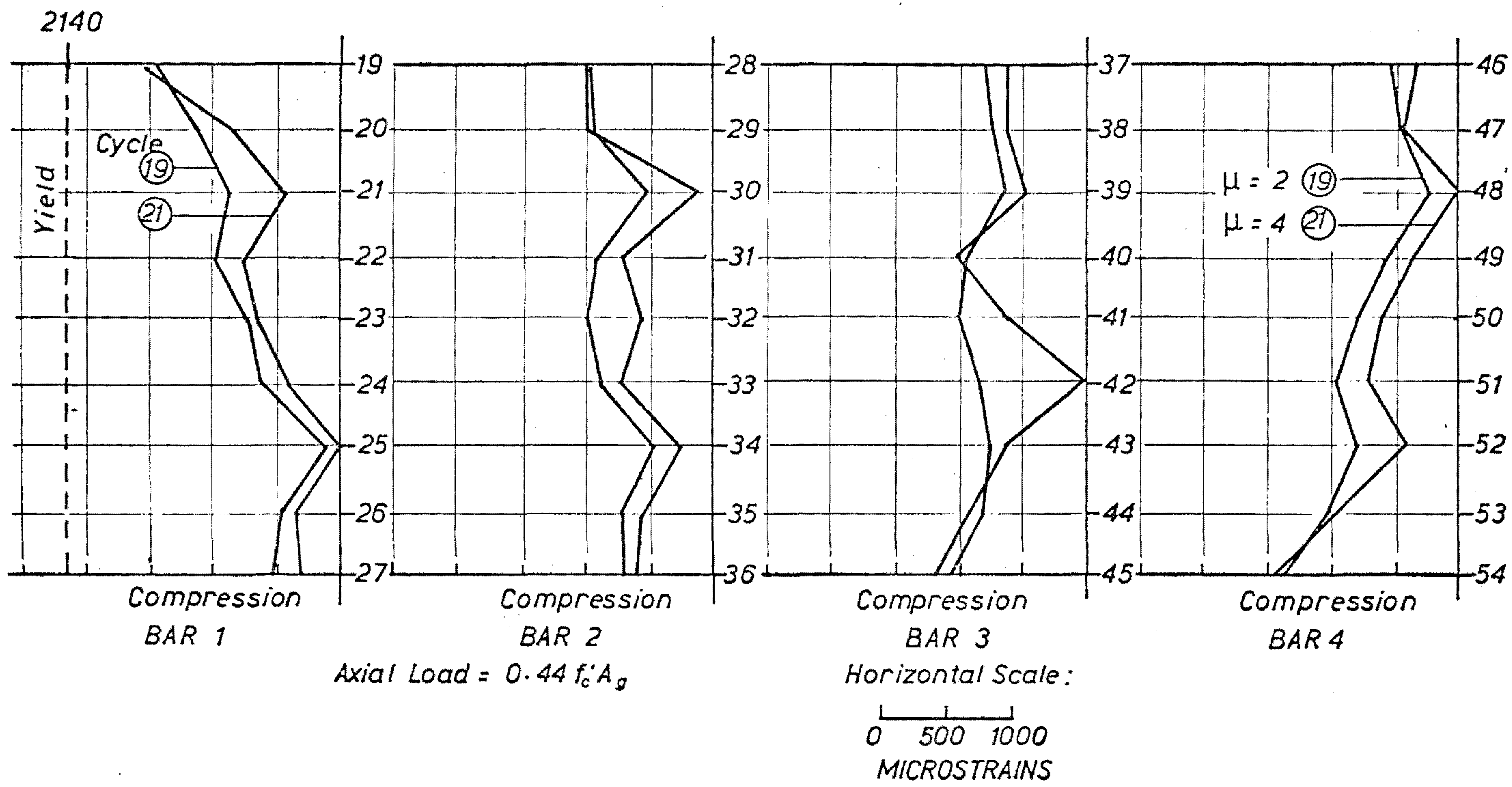


FIG. 5.19 COLUMN BAR STRAINS IN JOINT CORE OF UNIT B2

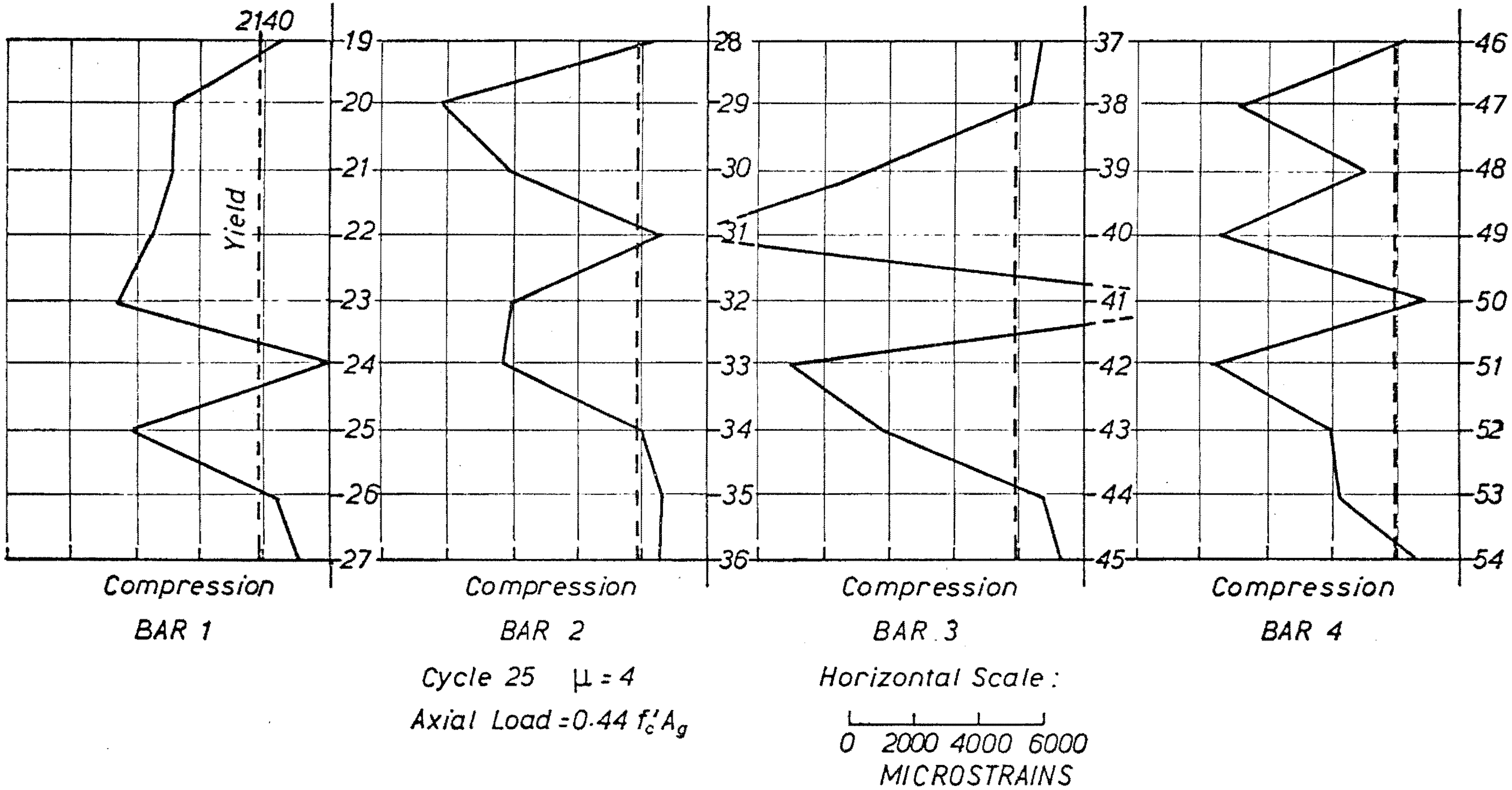


FIG. 5.20 COLUMN BAR STRAINS IN JOINT CORE OF UNIT B2

column where no confinement from the beam was possible; the beam being three quarters the width of the column.

5.3.3 A comparison of Units B1 and B2

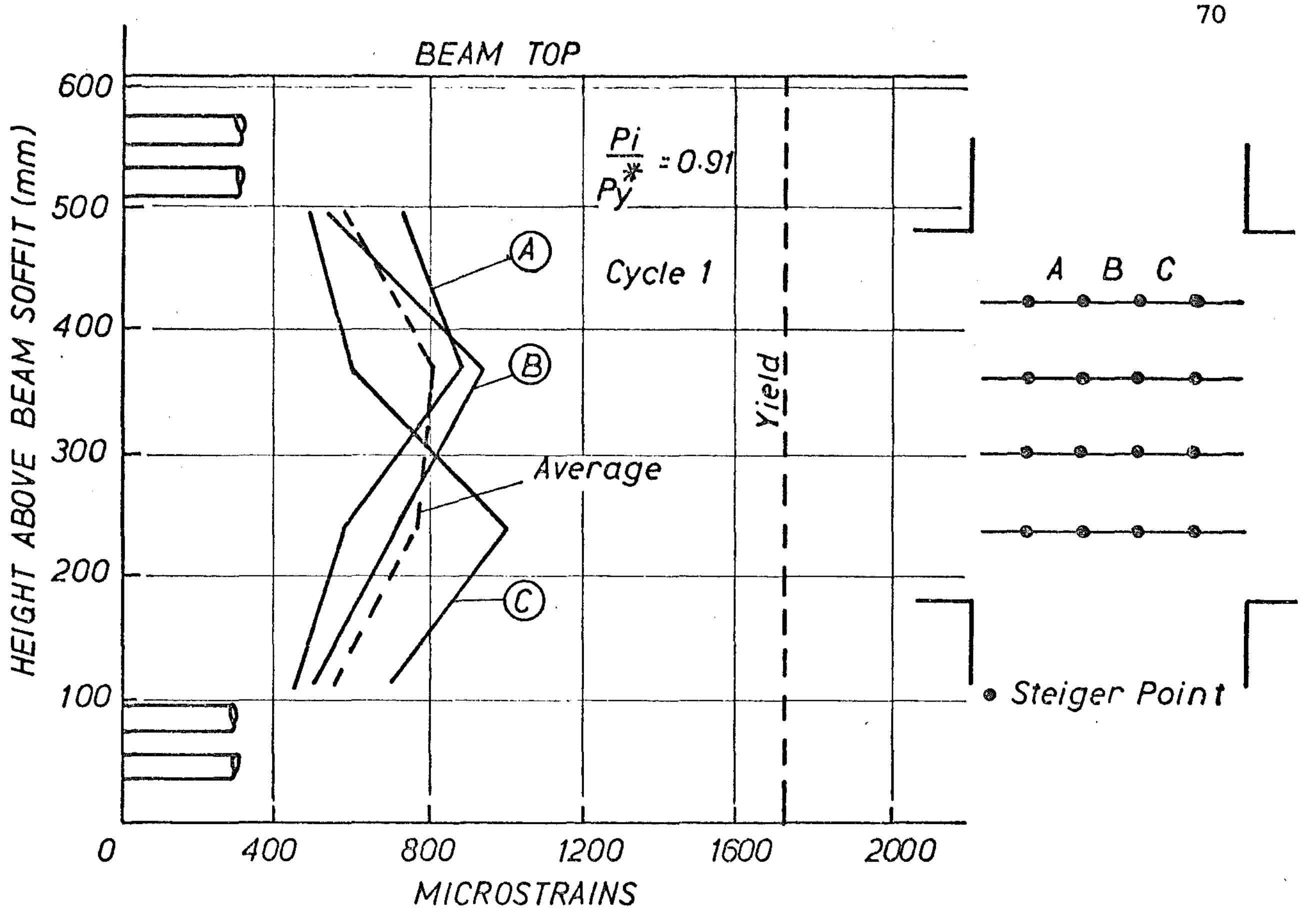
In both units the column reinforcement had induced strains significantly greater than theory would predict. In unit B1 and unit B2 the modular ratio (n) may have been higher than the value used. However, the strain peaks, especially in the intermediate column bars of unit B1, suggest these bars participate in resisting the shear forces induced in the joint. The column bars in unit B1 appear to be carrying tensile strains across the cracks in the joint zone. In unit B2 the compression field is dominant. This suggests that little contribution to the shear resistance of the joint, from restraint of the truss mechanism by the colum bars, is provided in the elastic cycles. High compression strains are induced in the inelastic cycles (Fig. 5.20). This is likely to be due to the compression failure in the joint, although column bar dowel action may have affected these readings.

5.4 JOINT STIRRUP-TIES

5.4.1 Unit B1

The longitudinal strain distributions for the exterior joint ties on the south face of the joint are plotted in Figs 5.21 and 5.22. The strain profile for each gauge location and also the average for each tie are given. The peak average strain for the last elastic cycle was 53% of the yield strain. The calculations in Appendix A indicate that there was at least 8% more horizontal shear reinforcement provided than that required. There is a certain amount of scatter along a tie set and a tendency for the ties located nearer the centre of the joint to be more highly stressed. This is more apparent in the inelastic cycles (see Fig. 5.22) than in the elastic cycles. The cracks in the elastic cycles were of similar width along the length of the failure plane whereas in the inelastic cracks the cracks were of greater width near the centre of the joint panel.

After cycle 14 at ductility of 4, all the ties had yielded with very large strains, especially in the centre ties. The ties had yielded due to excessive shear forces induced in the joint core. The shear resistance provided by the ties was insufficient, and the joint core



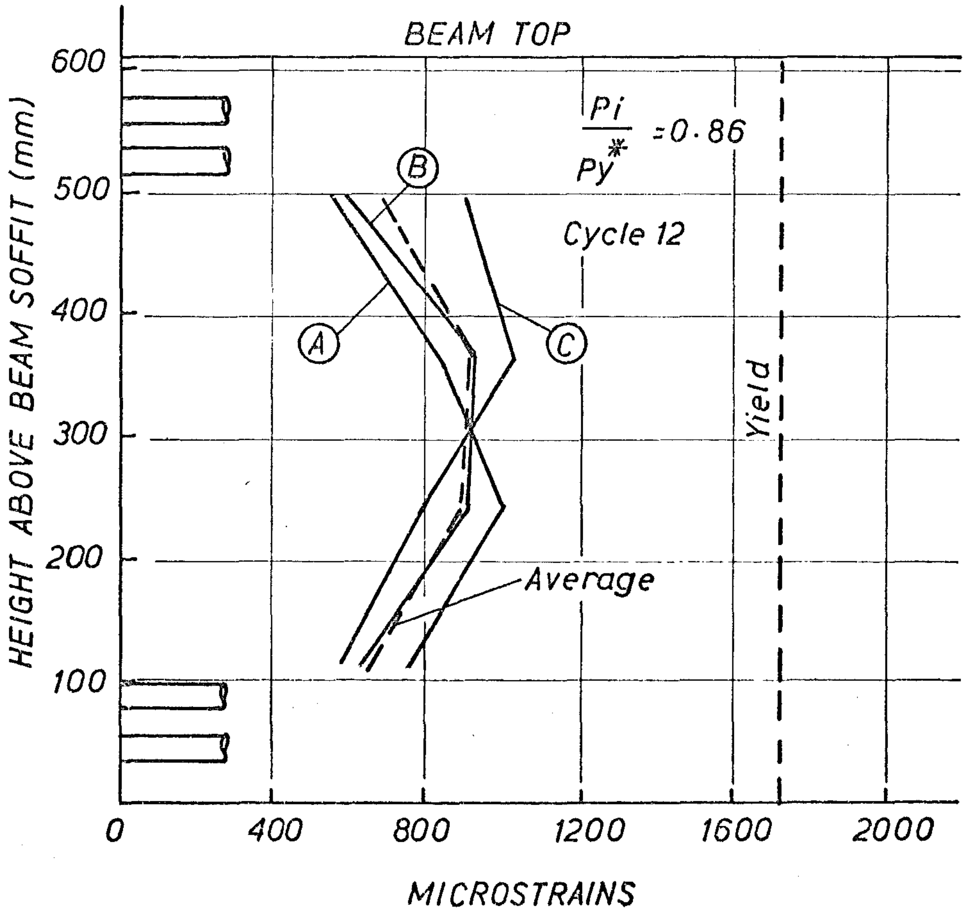
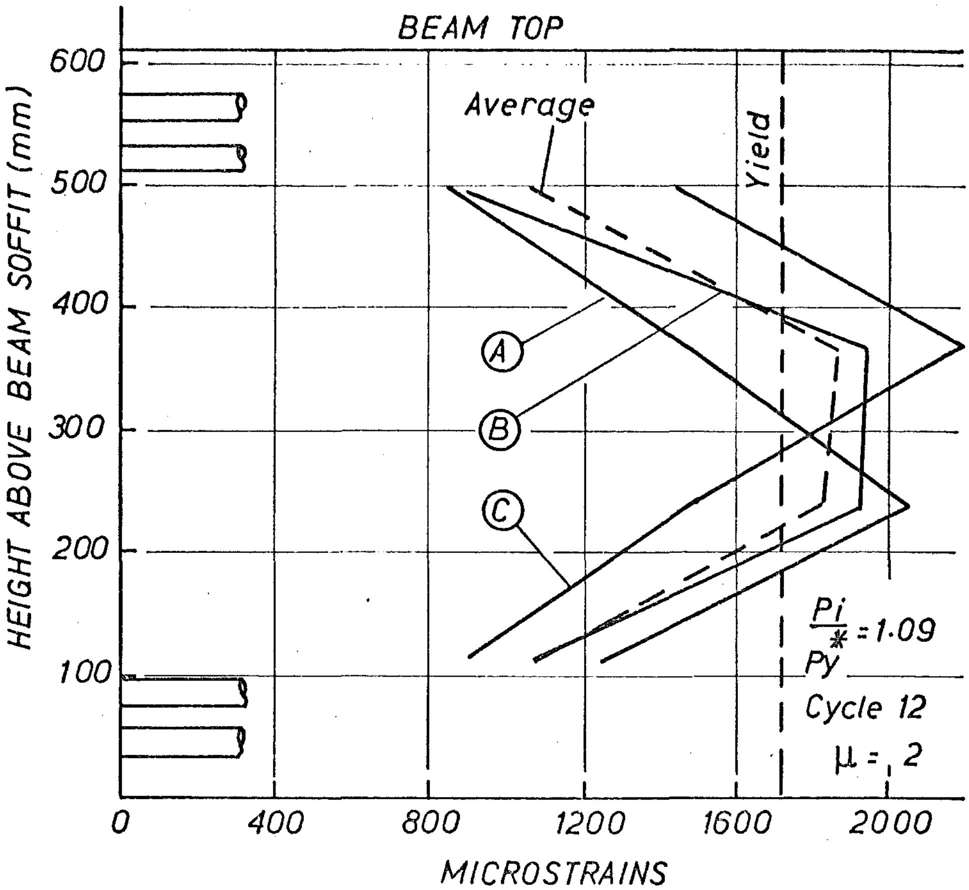


FIG. 5.21 JOINT TIE TENSION STRAIN DISTRIBUTION IN UNIT B1
ELASTIC LOAD CYCLES



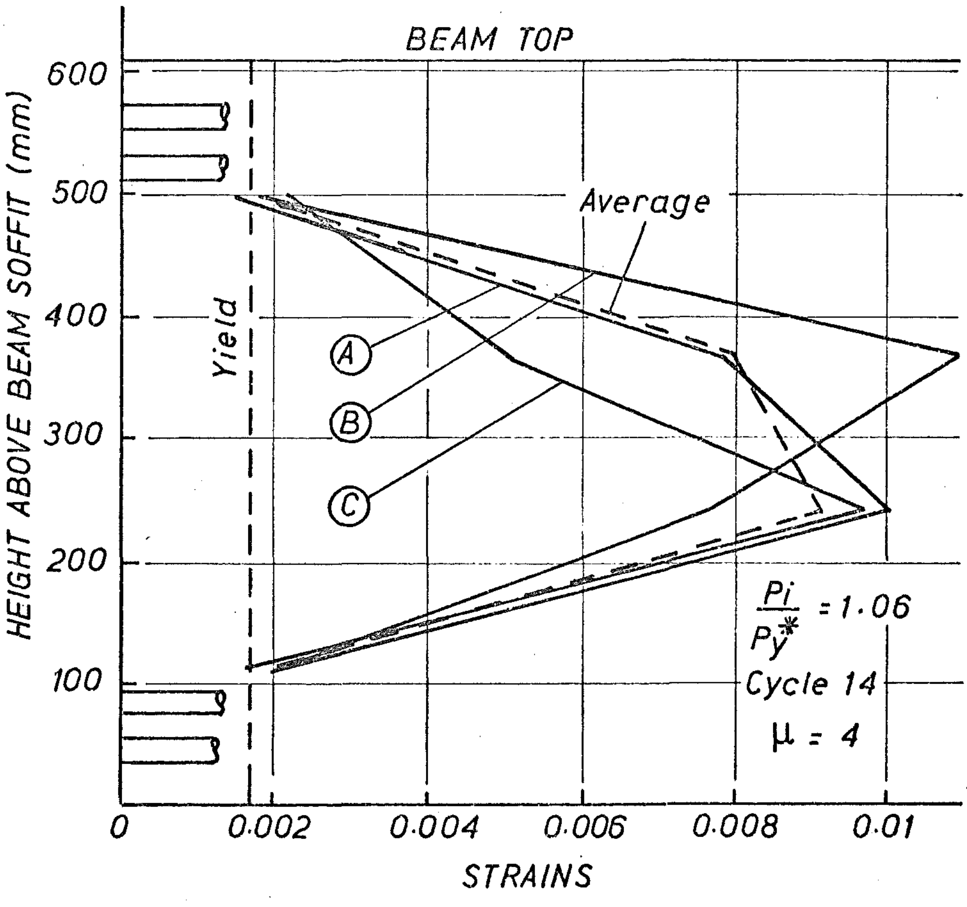


FIG. 5.22 JOINT TIE TENSION STRAIN DISTRIBUTION IN UNIT B1 INELASTIC LOAD CYCLES

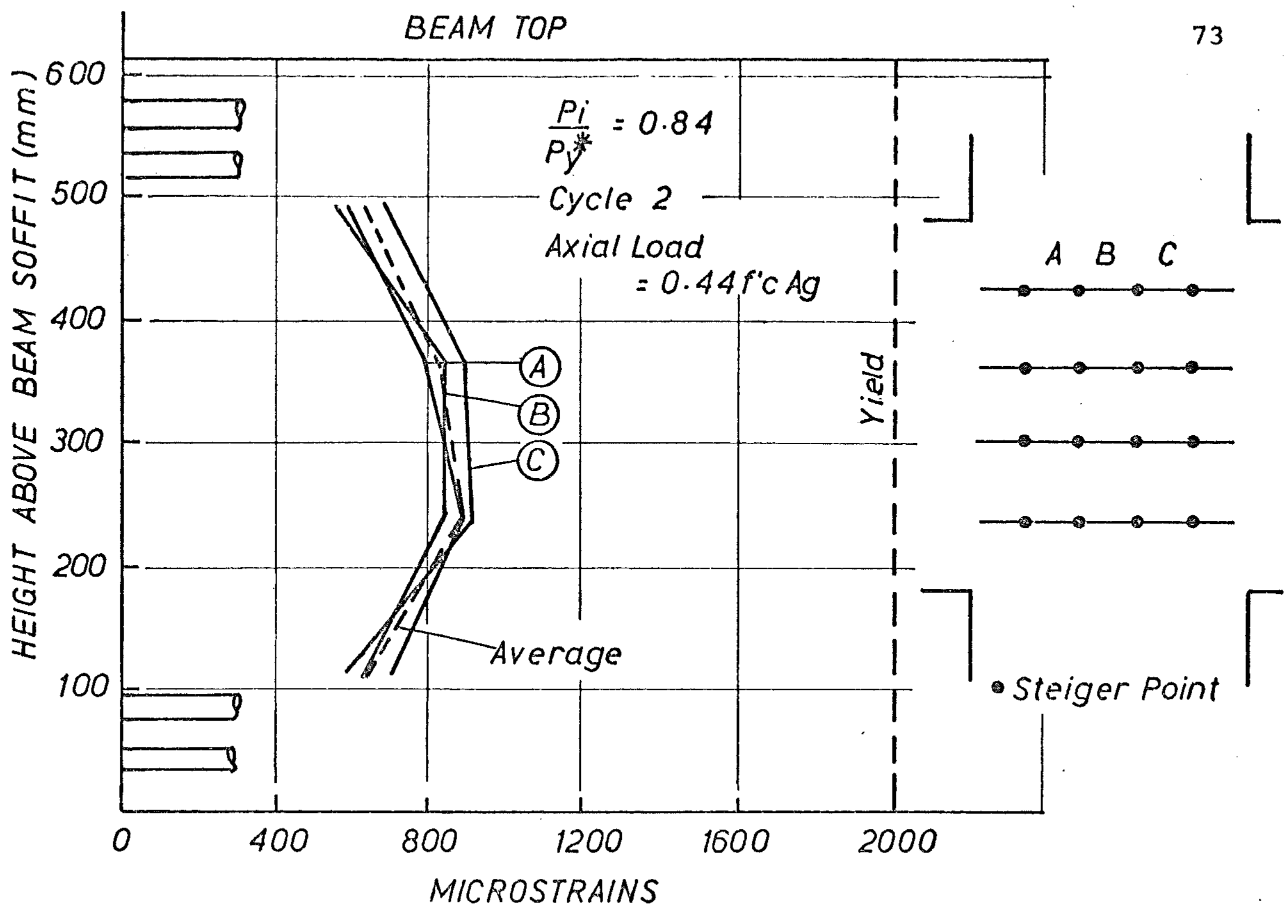
disintegrated after several inelastic cycles due to the breakdown of the concrete and steel shear resisting mechanisms.

5.4.2 Unit B2

The strain distributions for the joint ties are plotted in Figs 5.23 and 5.24. The maximum average strain at the high axial load, 2890 kN (0.44 $f_c' A_g$), was 52% of yield strain. The calculations in Appendix A indicated that the area of horizontal joint shear reinforcement provided for unit B2 was 13.5% less than that required. There was little increase in strain throughout the elastic cycles. With the lowering of the axial load to 2204 kN (0.34 $f_c' A_g$) the tie strains increased only slightly. When the axial load was lowered to 1645 kN (0.25 $f_c' A_g$) there was a larger scatter of strain readings between locations in a tie set and between the tie sets; the centre ties being more highly stressed. At this stage a maximum average strain of 70% of yield strain was measured. After the first inelastic cycle all the joint ties had yielded. Very large strains were recorded in the ties towards the end of the test and it was apparent that little shear resistance would have been provided by these ties at this stage. However, a substantial contribution to shear resistance would have instead been provided by dowel action of the column reinforcement.

5.4.3 A Comparison of Units B1 and B2

Both specimens performed satisfactorily in the elastic cycles with low strains being recorded in the outer ties. It is apparent that a small amount of degradation in the concrete shear resisting mechanism occurred. Moreover the strains measured in the ties did not increase by more than 16% during this initial part of the test. During the inelastic loading, all the ties in unit B2 yielded in the first cycles, whereas in unit B1 yielding of all the ties occurred only after the third cycle of inelastic loading. As Table 4.3 shows, the horizontal joint shear reinforcement in unit B1 was 3.82 times that provided in unit B2.



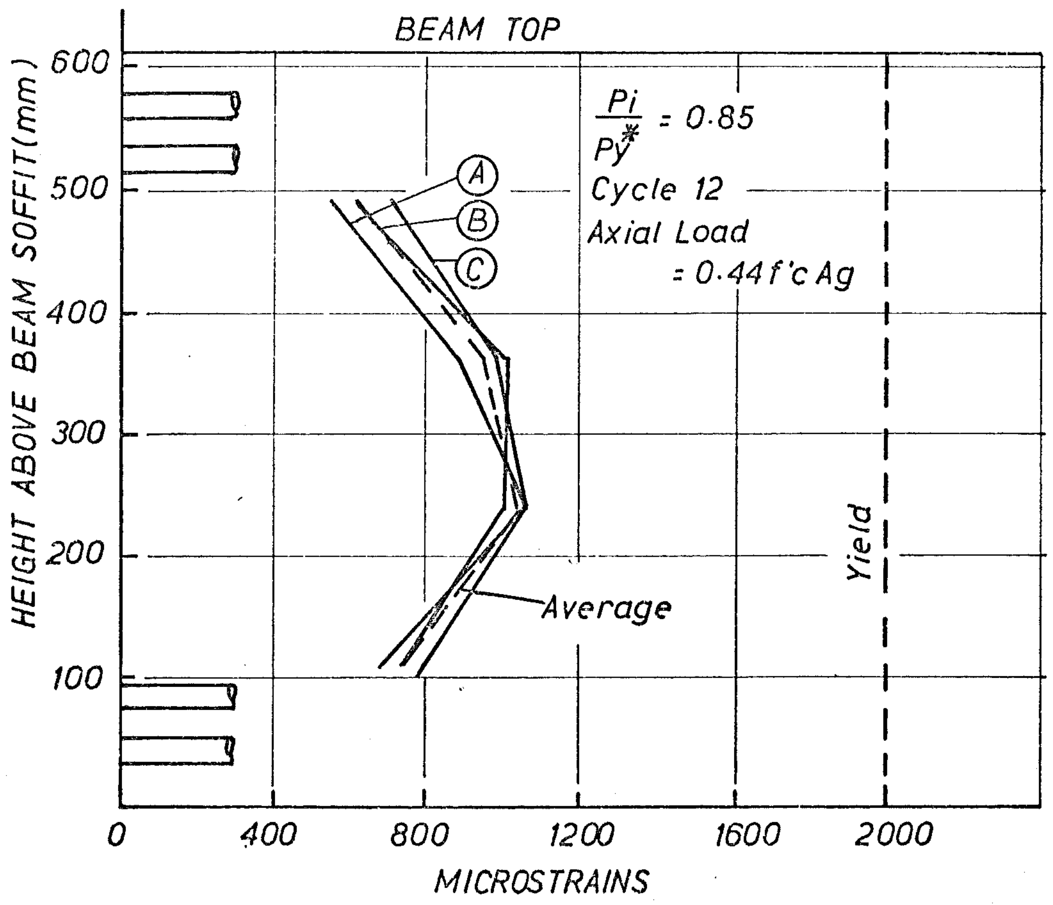


FIG. 5.23 JOINT TIE TENSION STRAIN DISTRIBUTION IN UNIT B2
ELASTIC LOAD CYCLES

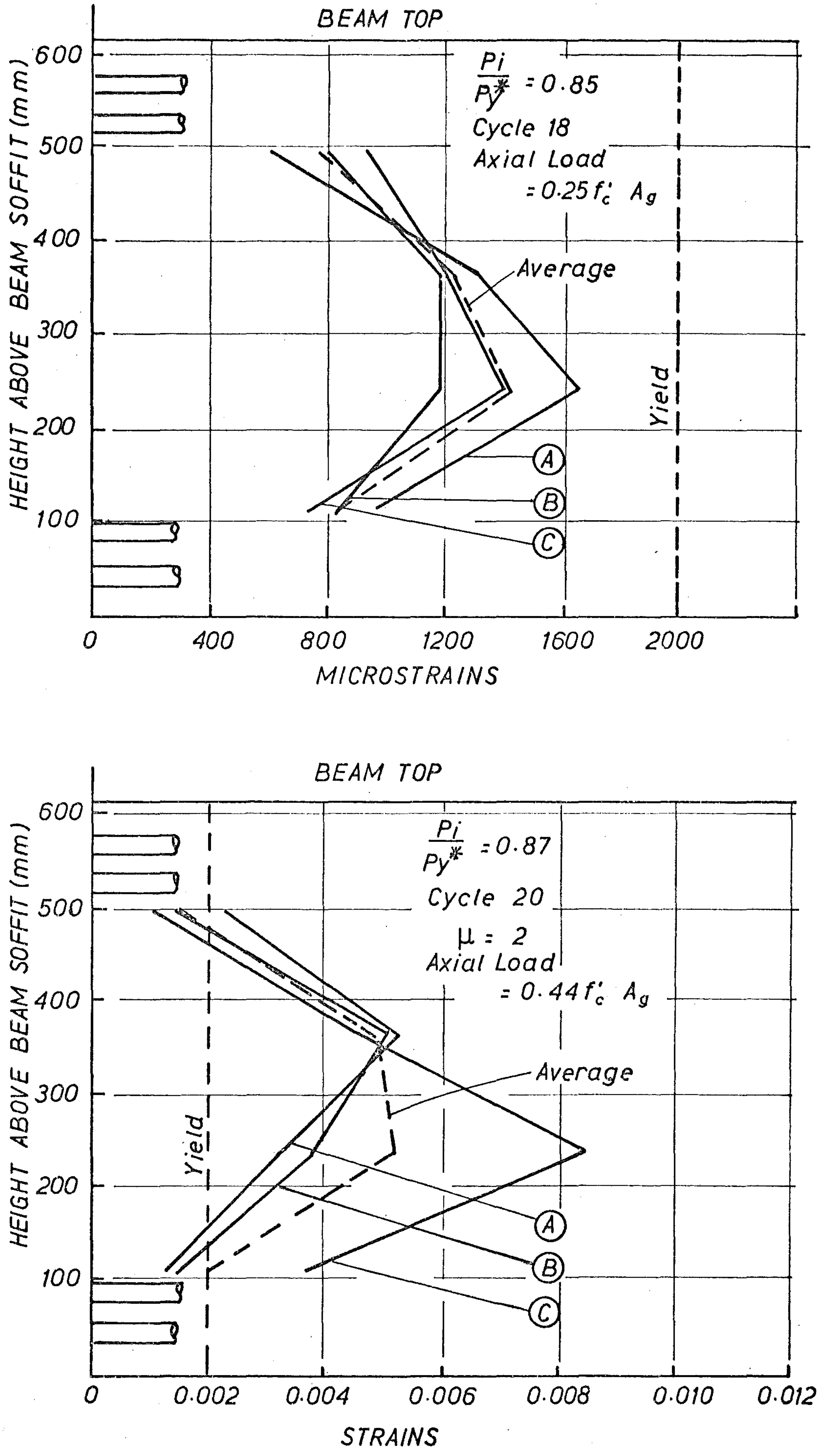


FIG. 5.24 JOINT TIE TENSION STRAIN DISTRIBUTION IN UNIT B2

CHAPTER SIX

THE INFLUENCE OF SOME VARIABLES ON ELASTIC JOINT BEHAVIOUR

6.1 INTRODUCTION

A number of variables that could influence the behaviour of an elastic beam-column joint are considered in this chapter. Three variables were chosen for case studies as they affect elastic joints. The results of these analyses are presented, and conclusions are drawn from them.

Factors Affecting Joint Behaviour

(a) Magnitude of axial load
(b) Horizontal joint shear reinforcement
(c) Confinement reinforcement in the joint
(d) Intersecting beams at the joint
(e) Vertical joint shear reinforcement
(f) Relative and absolute quantities of beam top and bottom flexual reinforcement content
(g) Joint aspect ratio
(h) Amount and distribution of column reinforcement
(i) Aggregate interlock
(j) Dowel action
(k) Bond transfer and yield penetration
(l) Diameter of beam and column bars
(m) Special joint steel devices

6.2 MAGNITUDE OF AXIAL LOAD

Axial compression can be expected to improve joint behaviour and reduce the demand for joint shear reinforcement. The mechanism of the diagonal concrete strut described in Chapter three indicates how axial compression can enhance joint behaviour. As suggested, a steeper diagonal concrete strut forms as a result of an enlarged compression block across the column section, and consequent reduction of the internal lever arm (Fig. 3.1). To maintain the inclination of the strut the share of the horizontal steel force, ΔT_C, that will combine with the total column compression stresses, must become larger (Fig. 3.6).

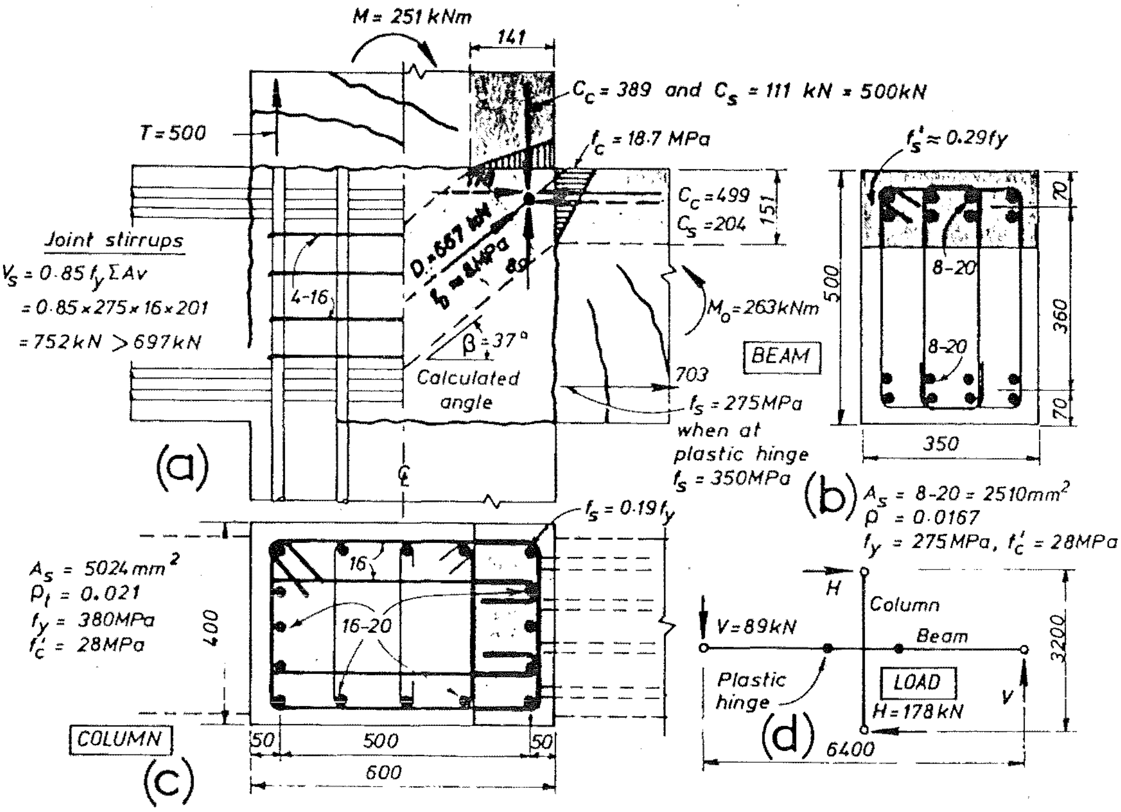


FIG. 6.1 THE DISPOSITION OF FORCES AT AN 'ELASTIC' BEAM-COLUMN JOINT

As indicated in Chapter three, a significant proportion of the horizontal and vertical shear force can be carried by this simple diagonal strut mechanism. Therefore analyses for an elastic joint were done, to determine the influence of the axial compression on the proportion of the horizontal shear force carried by the concrete mechanism, V_{ch}.

Fig. 6.1 is a typical beam-column joint unit used for the purpose of this analysis. The overall dimensions of the subassemblage are given in Fig. 6.1(d). The overstrength moment (M_o) at the column face of the beam, due to the removed plastic hinge, is such that it produces $f_s = f_y = 275$ MPa in the beam section indicated in Fig. 6.1(b). The design moment for the critical section of the column (Fig. 6.1(c)) was derived from M_o, so that the ideal strength of the column section corresponds with approximately 1.75 times the overstrength capacity of the critical beam section (i.e. $1.75 \times 251 = 439$ kN-m). Using elastic theory, the stresses in the concrete and steel for both the column and beam sections were found as indicated in Fig. 6.1(a) and (b). The horizontal shear force, V_{jh}, was calculated from equation (3.1) and the component of horizontal joint shear from the concrete shear resisting mechanism, V_{ch}, from equation (3.2). ΔT_c, as described in Chapter three, is the assumed bond force transferred from the beam steel to the surrounding concrete within the depth of the diagonal concrete strut, and it was calculated as

$$\Delta T_c = \gamma \; (A_s' \; f_s' + A_s \; f_y) \qquad (6.1)$$

An upper and lower bound for γ was chosen so that

$$2/3 \leq \frac{\gamma}{k_{col}} \leq 1.0$$

Fig. 6.1(a) indicates the magnitude of the diagonal concrete strut, i.e. D_c in Fig. 3.2, for the case of zero axial compression. The average diagonal compression on the strut is shown to be 8 MPa $\simeq f_c'/4$. The calculated inclination of the diagonal concrete strut, β, is shown in Fig. 6.1(a). The position of equilibrium of the internal horizontal and vertical forces contributing to the concrete shear resisting mechanism, D_c, is shown in Fig. 6.1(a) for the forces at the top right-hand corner of the free body. For simplicity, the intermediate column bars were not considered in the calculation of the column steel force at the critical section (i.e. C_s''' in Fig. 3.4). Diagonally opposite a similar position can be found from equilibrium of the internal horizontal and vertical forces contributing to D_c, at the bottom left-hand corner of the joint. For the case of equal top and bottom beam reinforcement the two points of equilibrium are in a similar location with respect to the boundaries of the joint. The diagonal compression strut must intersect these points of equilibrium as shown in Fig. 6.1(a). From the dimensions of the joint the inclination of the strut can be approximated. An alternative approach is to obtain β simply by considering all the internal horizontal and vertical forces as acting at the boundary of the joint core (Fig. 3.3). This assumed value of β is usually quite close to the value of β calculated from joint geometry when the centroids of the internal concrete and steel forces are located near the boundary of the joint core, as shown in Fig. 6.1(a). With axial compression on the column section the centroid of the concrete compression force in the column shifts towards the centre of the column. An 'exact' calculation of the line of thrust of the diagonal concrete strut would need to consider the position of this column concrete compression force as well as the forces in, and the positions of the intermediate column bars. However, the simplifications involved in postulating these mechanisms do not warrant a high degree of accuracy.

The horizontal joint shear force assumed to be carried by the truss mechanism, V_{sh}, was obtained from equation (3.3). The area of horizontal shear reinforcement, A_v, was calculated using an under-capacity factor (ϕ) of 0.85, as shown in Fig. 6.1(a).

The vertical component of the joint shear force, V_{jv}, was estimated from V_{jh} and β. This enabled calculation of the vertical shear force to be transferred by the truss mechanism, V_{sv} (equation 3.5). This vertical shear force was assumed to be provided by intermediate vertical column bars only. The area of the intermediate bars required was calculated in a similar manner to that for the horizontal joint stirrups (Fig. 6.1(a)).

At a column design axial load, (N_u), of 0.3 $f_c' A_g$, the neutral axis depth for the column section is approximately equal to the column depth (h_c). The column was analysed for axial compression loads between zero and 0.3 $f_c' A_g$. This enabled the proportion of the total horizontal joint shear carried by the concrete mechanism, V_{ch}/V_{jh}, to be calculated for different axial loads. Fig. 6.2 represents a plot of V_{ch}/V_{jh} versus the axial load index $N_u/f_c' A_g$.

The proportion of shear carried by the concrete mechanism increases approximately linearly as the axial load increases. Fig. 6.2 indicates that the assumption for γ is more significant at high axial loads in assessing the value of V_{ch}. For the particular example chosen (Fig. 6.1) it appears that approximately 40% of the total horizontal joint shear may be carried by the concrete shear resisting mechanism when the applied axial load on the column section is zero, and the ratio of top to bottom beam reinforcement (ρ/ρ') is 1.0. By extrapolation of the curve in Fig. 6.2, with $\rho/\rho' = 1.0$ and $\gamma = 2/3\ k_{col}$, it is apparent that all of the horizontal shear in the elastic joint may be transferred by the concrete mechanism when the axial compression is 0.5 $f_c' A_g$. When $\rho/\rho' = 0.5$ and $\gamma = 2/3\ k_{col}$ it is similarly estimated that for zero axial load $V_{ch}/V_{jh} = 0.28$, and for axial compression of 0.66 $f_c' A_g$, $V_{ch}/V_{jh} = 1.0$. These appear to be conservative estimates.

The recommendations of the New Zealand National Society for Earthquake Engineering (NZNSEE)[9] for elastic joints, discussed in Chapter two, are also plotted on Fig. 6.2. The method of analysis, to take into account the effect of the variation in the ratio of beam bottom and top reinforcement content, is explained in section 6.7.

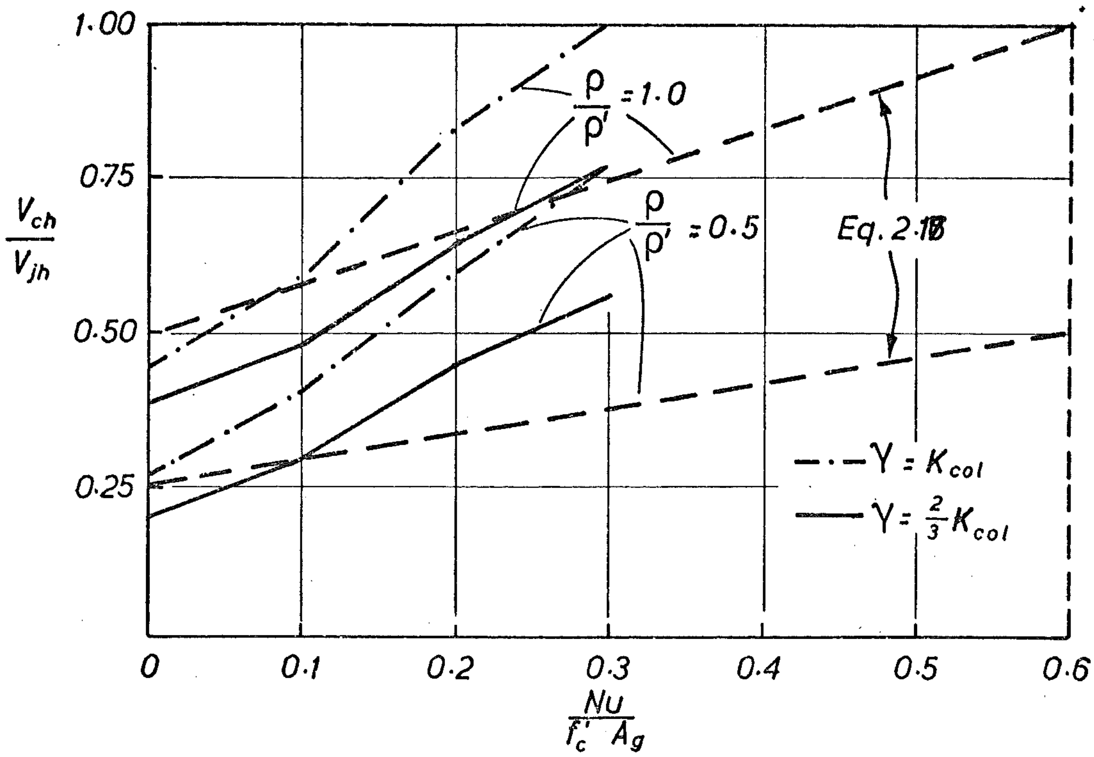


FIG. 6.2 CONTRIBUTION OF DIAGONAL STRUT ACTION TO HORIZONTAL JOINT SHEAR RESISTANCE VERSUS AXIAL LOAD

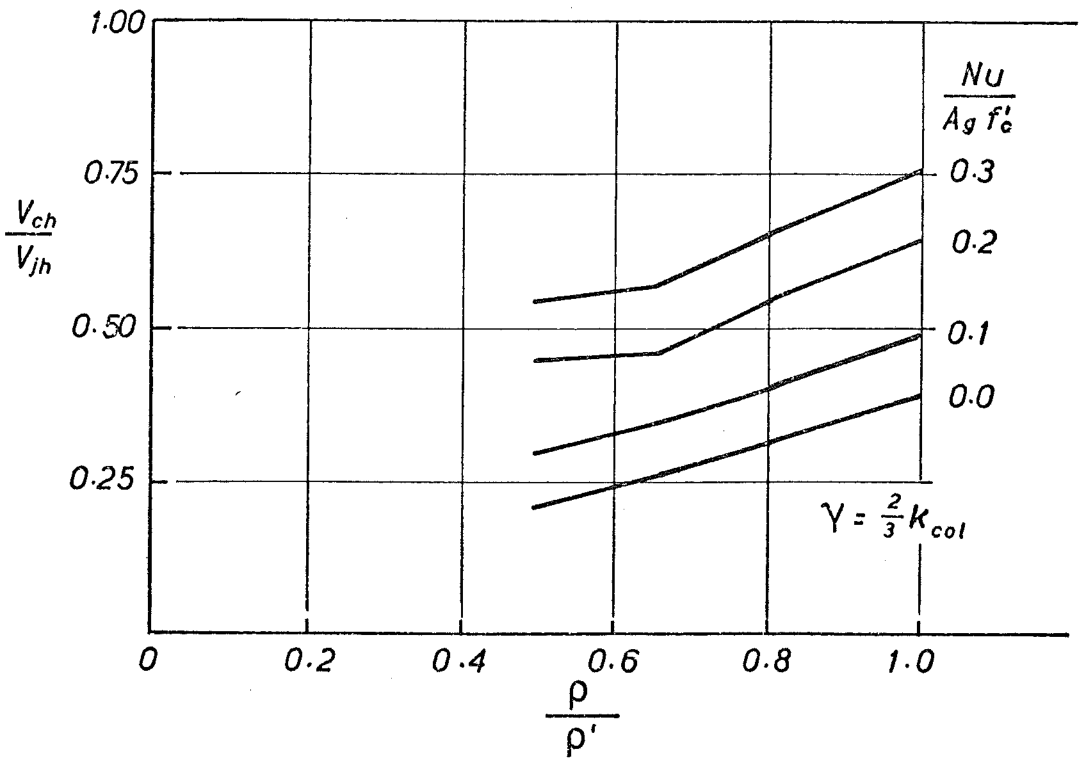


FIG. 6.3 CONTRIBUTION OF DIAGONAL STRUT ACTION TO HORIZONTAL JOINT SHEAR RESISTANCE VERSUS REINFORCEMENT RATIO

The NZNSEE recommendations[9] for $\rho/\rho' = 1.0$ and 0.5 appear to be of the right order. According to the analysis presented here, the assumption that $V_{ch}/V_{jh} = 0.5$ when $N_u = 0.0$ may be unconservative, whereas the assumption that $V_{ch}/V_{jh} = 1.0$ when $N_u = 0.6\, f_c'\, A_g$ may be too conservative.

The results, for the test units discussed in Chapter six, indicate, however, that the suggested design approach, using the elastic model, is satisfactory and conservative. For this reason, little adjustment in the NZNSEE design curve appears necessary or justified. A better estimate of the value of V_{ch} in equation (2.17) is given by

$$V_{ch} \leq \frac{V_{jh}}{2.2}\left(1 + \frac{N_u}{0.5\, A_g f_c'}\right) < V_{jh} \qquad (6.2)$$

6.3 HORIZONTAL SHEAR REINFORCEMENT

Up to a limit it has been found that the greater the amount of transverse shear reinforcement in the joint, the greater the shear strength. Researchers[5,14] have found, however, that the increase in shear strength was not proportional to the shear reinforcement used. It is also not clear how effective different shapes and arrangements of stirrups are. For design purposes it is suggested[9] that the effective area of each stirrup tie set, that crosses the critical failure plane, be determined according to the orientation of the individual tie legs with respect to this failure plane.

6.4 CONFINEMENT REINFORCEMENT

Increased transverse reinforcement perpendicular to the direction of applied shear, acting as confining reinforcement, would be expected to increase the compression carrying capacity of the concrete in the core of an inelastic joint after extensive damage. In the elastic domain, confining reinforcement would not be effective. Present recommendations[1,9] suggest that the same confinement, as provided in potential plastic hinge regions of columns, be used in the joint region.

As discussed in Chapter two the NZNSEE[10] approach is different to that of ACI-ASCE 352.[1] The first method attempts to ensure that adequate ductility may be achieved in the potential plastic hinge region, and considers such variables as the level of axial load on the column, the longitudinal steel ratio, the proportion of the column section confined, and the stress-strain relationship of the longitudinal steel and the confined concrete. The second method is based on preserving the axial load strength of the column after the cover concrete has spalled.

6.5 INTERSECTING BEAMS AT THE JOINT

Intersecting beams, perpendicular to the direction of applied shear, may aid confinement of the joint core. The ACI-ASCE 352[1] approach is to increase the allowable concrete shear stress in the joint by 40% where the intersecting beams cover sufficient area of the joint region. This is applicable to both Type 1 and Type 2 joints.

The NZNSEE[9] approach is to consider increased confinement from the transverse beams only when no yielding of the beam reinforcement in these transverse beams is likely. If this is the case, then the confinement reinforcement required[10] may be reduced by one half, where there is sufficient coverage of the joint by the beams.

It is not suggested that any significant increase in the proportion of the total joint shear carried by the concrete will occur in an elastic joint confined by beams. The compression field in a joint responding elastically is relatively lowly stressed. No alteration to the joint shear mechanisms, discussed in Chapter three, appears to be justified.

6.6 VERTICAL SHEAR REINFORCEMENT

The use of vertical shear reinforcement would, within limits, increase the shear strength of a joint. The reinforcement may be provided by intermediate column bars proportioned to resist forces in addition to flexural requirements. Special bars passing through the joint and adequately anchored outside the joint region, or vertical stirrup ties, may be used as vertical shear reinforcement.

It appears that no experimental work has been done on beam-column joints with intentionally designed vertical shear reinforcement. Therefore calculations for the amount of vertical shear reinforcement required lack experimental basis. Where column reinforcement passes through a joint, it is difficult to isolate the components of the bar force, one component being the flexural bar force transferred across the joint, the other results from vertical shear in the joint.

However, present research[18] at the University of Canterbury investigated the effect of intermediate column bars on the seismic behaviour of beam-column joints. These test units consisted of prestressed beams and reinforced concrete columns. The inclusion of intermediate column bars appeared to result in a dramatic improvement in joint performance compared to similar units previously tested[15] with no intermediate bars.

6.7 RELATIVE AND ABSOLUTE QUANTITIES OF BEAM TOP AND BOTTOM REINFORCEMENT

The relative quantity of beam top and bottom reinforcement (ρ/ρ') may be expected to influence the behaviour of an elastic joint. This can be explained with reference to the elastic model reviewed in Chapter three. It is evident that the diagonal concrete strut (Fig. 3.2) is a more efficient mechanism for shear transfer than the truss mechanism (Fig. 3.3) of the joint core. With reference to Fig. 3.2, for equal quantities of beam top and bottom reinforcement, the internal concrete compression forces at ultimate in the beam on either side of the joint are the same, i.e. $C_c = C_c'$ The diagonal concrete strut is formed between the compression zones of the beam and column sections. If the ratio of top to bottom beam reinforcement was greater than unity, the value of C_c' on the left-hand side of the joint core would be smaller than the corresponding concrete compression force C_c at the right-hand side. With the smaller compression zone, in one of the beam sections at the face of the joint core, the force in the diagonal concrete strut would obviously become smaller. In the region of the top reinforcement a greater proportion of the horizontal shear force would need to be carried by the truss mechanism. Therefore the analysis becomes more indeterminate.

Analyses of the prototype beam-column joint, shown in Fig. 6.1(a), were carried out with the top reinforcement content being kept constant as indicated in Fig. 6.1(b). However, the bottom to top reinforcement content ratio (ρ/ρ') was varied from 1.0 to 0.5.

The approach was similar to that described in section 6.2(a). The new horizontal joint shear carried by the diagonal concrete strut, V_{ch}, was calculated from equation (3.2) and the bond force, ΔT_c, was calculated from equation (6.1). In this $\gamma = 2/3\ k_{col}$.

With $\rho/\rho' = 0.5$, the beam compression force on the right-hand side of Fig. 6.1(a) was close to one half of the beam compression force for the corresponding situation with equal top and bottom reinforcement. The overstrength moment (M_o) on that half of the beam-column assembly was also one half of that shown in Fig. 6.1(a), as it was assumed that the bottom reinforcement was at yield, i.e. $f_s = f_y = 275$ MPa.

In evaluating the load carried by the diagonal strut, the smaller of the beam flexural compression forces, i.e. that based on $\rho' < \rho$ was used. From this the proportion of the horizontal shear force assigned to the concrete mechanism was calculated. It should be noted that all other properties being equal when $\rho/\rho' = 0.5$, the total horizontal joint shear force is approximately three quarters the total horizontal joint shear force with $\rho/\rho' = 1.0$.

The axial load was varied as in section 6.2(a) for each value of ρ/ρ'. The plots of V_{ch}/V_{jh} versus ρ/ρ' are shown in Fig. 6.3. It can be seen that for $\rho/\rho' = 0.5$, with zero axial load, the value of V_{ch}/V_{jh} is 54% of the value for $\rho/\rho' = 1.0$. The above approach for determining the value of V_{ch} is likely to be conservative.

The absolute quantity of beam reinforcement has been shown to be significant in beam-column joint units responding inelastically in the joint core. Unless the flexural tension reinforcement content in the plastic hinge regions of beams is kept small, i.e. less than approximately 1.5%, the horizontal joint stirrup reinforcement required can result in serious congestion of the joint. The problem of bar slip can also arise. With an elastic joint, the transfer of bond forces is much more favourable and less joint reinforcement is required. This would enable a greater quantity of beam steel to be used through the joint.

6.8 JOINT ASPECT RATIO

Tests on deep beams with span/depth ratios less than 2, where arch action is the dominant mode of shear resistance, have indicated that as the span/depth ratio becomes smaller the shear strength of the reinforced concrete beam increases. In investigations of beam-column joints, few researchers appear to have tested units with different aspect ratios. Meinheit and Jirsa of the University of Texas[5] have reported tests on beam-column joint units with two different aspect ratios. The aspect ratio for these tests was defined as h_c/h_b, where

h_c is the depth of the column, and h_b is the depth of the beam, both measured in the plane of the applied beam shear. The aspect ratios used by them were 0.72 and 1.0. Meinheit and Jirsa report that the aspect ratio did not appear to have affected the ultimate shear strength of the test units although the units with the smaller aspect ratios, and greater column width, generally had a greater shear capacity after the completion of four inelastic cycles.

In an elastic joint, a decrease in the aspect ratio would appear to be accompanied by an increase in the angle of inclination, β, of the concrete shear resisting mechanism (Fig. 3.2). This assumes that the internal forces and the zones of compression acting at the joint boundary remain unchanged. This increase in inclination would require a greater contribution from the horizontal force $(C_c + \Delta T_c^* - V_{col})$ (Fig. 3.6) to maintain equilibrium, implying the horizontal joint shear carried by the concrete mechanism would increase.

Calculations were done for several elastic beam-column joints, using the loading system in Fig. 6.1(d), with different joint aspect ratios, to determine the effect on the horizontal component, V_{ch}, of the diagonal concrete strut. The aspect ratio of the beam-column joint was defined in terms of the joint core dimensions as h_{cj}/h_{bj}, where h_{cj} is the length of the joint core between the centre of reinforcement in the compression face of the column and the centre of reinforcement in the tension face. The depth of the joint core h_{bj} is the corresponding beam depth between centres of beam reinforcement. The ratio of the core to overall dimensions, defined as h_{cj}/h_c and h_{bj}/h_b, was taken as 0.8. The ratio of the amount of top to bottom flexural reinforcement in the example beam was taken as being equal. The area of the beam reinforcement was the same for every beam section ($A_s = A_s' = 2510$ mm^2 as in Fig. 6.1(b)).

The axial load on the column section was zero and the beam overstrength moment, M_o, was such that $f_s = f_y = 275$ MPa at the column face. From equilibrium of the system (Fig. 6.1(d)) the appropriate column moment on the section at the beam face could be found. The design moment for the critical column section was derived from M_o (see section 7.2), so that the ideal strength of the column section corresponded with approximately 1.65 times the overstrength capacity of the critical beam section.

The joint shown in Fig. 6.1(a) has an aspect ratio of 1.2. For this analysis the aspect ratios were chosen so that $0.5 < h_{cj}/h_{bj} < 2.0$.

Two methods for varying the joint aspect ratio were used. It was decided for the first method to use a constant joint shear area as a means of determining the dimensions of the column and beam sections. The shear area in this case would be defined as the length of the column times the width of the beam ($h_c b_w$). In the second method the beam dimensions were kept the same in every case, and the only variation was in the depth of the column. As in Fig. 6.1(c), the width of the column section (b_c) was taken as being 50 mm wider than the beam width (b_w) for both methods.

Table 6.1 shows the beam and column dimensions, the beam and column reinforcement percentages, and the aspect ratios used in method 1.

Table 6.1 Joint Parameters

	h_b (mm)	b_w (mm)	$\rho = \rho'$	h_c (mm)	b_c (mm)	ρ_t	$\frac{h_{cj}}{h_{bj}}$
1	700	600	0.007	350	650	0.050	0.5
2	667	525	0.009	400	575	0.046	0.6
3	625	420	0.011	500	470	0.030	0.8
4	500	350	0.016	600	400	0.023	1.2
5	625	420	0.020	500	470	0.015	1.5
6	350	300	0.026	700	350	0.010	2.0

Using elastic theory, the stresses in the column and beam sections at the joint boundaries were calculated as shown in Fig. 7.1(a). From equation (3.2) the proportion of the total horizontal joint shear carried by the concrete shear resisting mechanism was calculated, and results of V_{ch}/V_{jh} versus h_{cj}/h_{bj} are plotted in Fig. 6.4 for both methods. The value of γ in equation (6.1) was taken as 2/3 k_{col}. The joint parameters for the second method are shown in Table 6.2. For the second method of analysis the aspect ratio was chosen between 0.8 and 1.8, as within these bounds the column reinforcement content was found to be within the recommended[3] limits.

From Fig. 6.4 it is seen that there is an increase of approximately 40% in the horizontal joint shear carried by the concrete shear resisting mechanism when the joint aspect ratio is 0.6 compared to the case when it is 2.0. The two methods of analysis give similar results, and the actual shear area of the joint does not appear to affect the calculations of V_{ch}/V_{jh}.

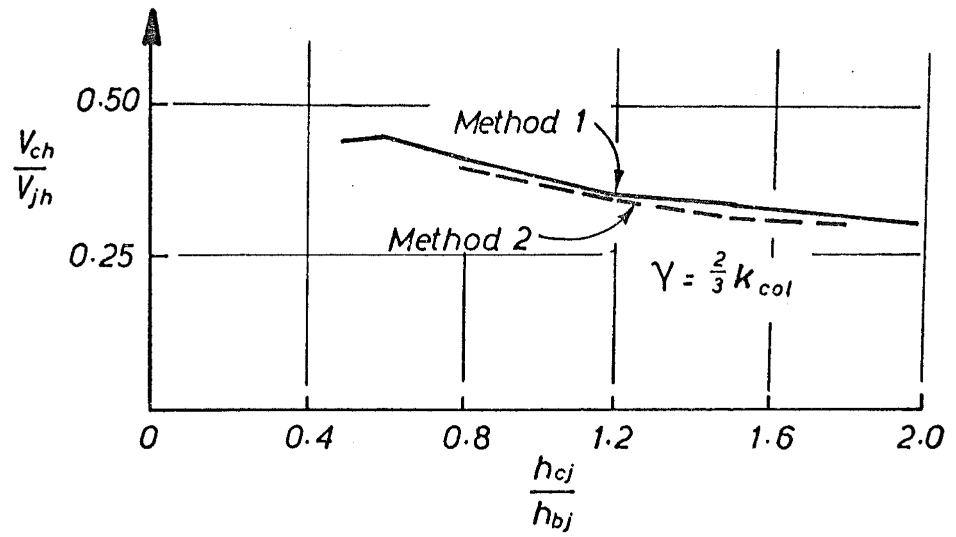


FIG. 6.4 PROPORTION OF HORIZONTAL SHEAR FORCE RESISTED BY CONCRETE MECHANISM VERSUS JOINT ASPECT RATIO

Table 6.2 Joint Parameters

	h_b (mm)	b_w (mm)	$\rho = \rho'$	h_c (mm)	b_c (mm)	ρ_t	$\frac{h_{cj}}{h_{bj}}$
1	500	350	0.0167	400	400	0.059	0.8
2	500	350	0.0167	750	400	0.014	1.5
3	500	350	0.0167	900	400	0.010	1.8

6.9 AMOUNT AND DISTRIBUTION OF COLUMN REINFORCEMENT

The amount of column reinforcing bars passing through a beam-column joint has been found by some researchers[5] to have little influence on the performance of a joint. The test units reported would appear to have been more than adequately reinforced with respect to vertical shear in terms of section 6.6. As previously stated in that section, the vertical reinforcing would only enhance shear performance up to a limit. The above test units were reinforced with column reinforcement percentages (ρ_t) of between 2.05% and 6.7%.[5]

After only two cycles into the inelastic range, nearly all the test units suffered a joint failure because of insufficient horizontal shear reinforcement. With this type of joint failure occurring[5] it is not surprising that variations in the column reinforcing percentage were not a significant parameter.

Flexural reinforcing content in columns can be as low as 1%[3], and in this case vertical joint shear may become critical. In tests at the University of Canterbury[14,15] columns of the test specimens were lightly reinforced, as low as 1.25%, with no intermediate bars. It is now considered[18] that one possible cause of unsatisfactory joint performance for these units, was the absence of intermediate column bars and of high axial load to resist the vertical shear.

An even distribution of column bars along the face of the joint, in the plane of the shear force (see Fig. 4.2) is thus considered[9] as essential. Column bars located near the tension and compression faces of the column are not likely to be effective in resisting the vertical joint shear forces.

6.10 AGGREGATE INTERLOCK

Present knowledge[2] on the shear behaviour of simply supported beams with rectangular cross sections indicates that a substantial contribution to the shear resistance of such beams may be provided by aggregate interlock. Beam action, as opposed to arch action, is generally observed with shear span/depth ratios of greater than 3, although the transition from the one mode of shear resistance to the other is gradual. For aggregate interlock to occur in concrete, there has to be shear displacement along a failure plane, or, as in the case of beam action, shear displacement along an inclined crack. Flexure of the beam causes relative movements along these cracks and this leads to aggregate interlock, provided that crack widths are kept relatively small.

In the case of a beam-column joint several points should be mentioned. Firstly the aspect ratios of most beam-column joints are less than 2.0, and arch action predominates. Secondly the cracks that form in beam-column joints are different to those occurring in a reinforced concrete beam. The diagonal tension cracks in a beam are, with few exceptions, extensions of flexural cracks. In beam-column joints the diagonal tension is due to shear and the splitting effect of the diagonal compression.

For aggregate interlock to occur, a relative displacement along these diagonal cracks would be needed and it is not evident that this would eventuate.

The shear forces introduced to the boundaries of the joint are as indicated in Figs 3.2 and 3.3. No shear displacement along the potential failure plane appears possible unless distortions, other than those due to pure shear, of the joint core at the boundaries occur. In joints responding inelastically to cyclic loading, deterioration of the joint core may be caused by yielding of the joint reinforcement. Uneven bearing of the concrete, after the closure of the cracks, may lead to relative shear displacements causing some aggregate interlock.

In an elastic joint it is suggested that the shear displacement along the diagonal cracks that run approximately parallel to the potential failure plane is negligible and aggregate interlock is insignificant.

6.11 DOWEL ACTION

Dowel action has also been identified as one of the shear resisting mechanisms in beams. For dowel action to occur in a beam-column joint shear dispacements, transverse to the bars passing through the joint, are required. The occurrence of shear displacement along a shear failure plane, as discussed in section 6.10, may be sufficient to provide some dowel action. A more significant source of transverse displacement, affecting both the column bars, beam bars, and also the horizontal stirrup ties, is possible when the diagonal cracks open up (Fig. 6.5). The magnitude of the horizontal and vertical components of this diagonal displacement depends upon the angle of inclination of the crack. For column bars intersecting these cracks the horizontal component of the diagonal crack will induce a transverse shear displacement in the joint core. This may be accompanied by a kinking of the column bars across the crack (Fig. 6.5). The beam bars and horizontal stirrup ties may also be subject to dowel shear when a transverse shear displacement is produced from the vertical component of the diagonal crack.

Such dowel action may be a major source of shear resistance when all the other sources are exhausted, i.e. when yield of the joint reinforcement and deterioration of the diagonal concrete strut have taken place.

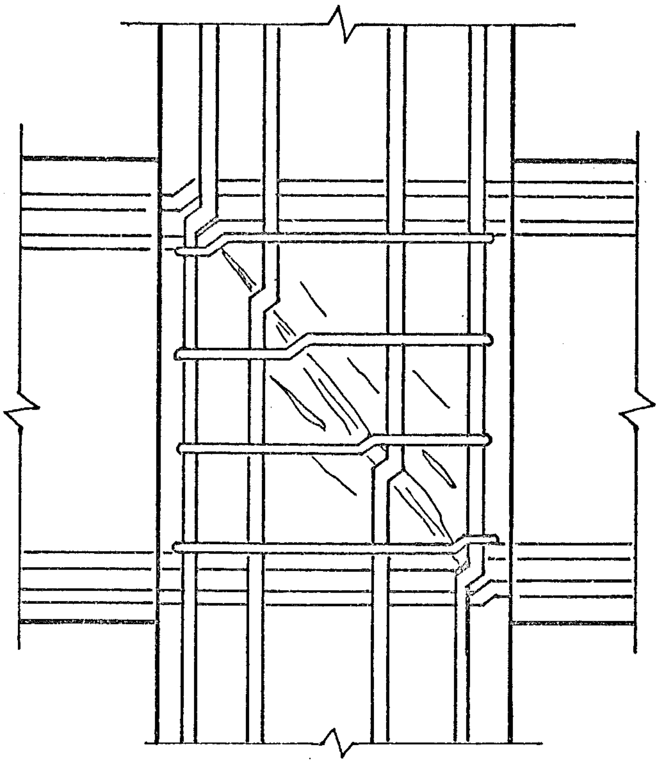


FIG. 6.5 DOWEL ACTION IN A YIELDING JOINT

The large deformations associated with this mechanism would be accompanied by severe stiffness degradation. For elastic joints crack widths must remain quite small and hence dowel action should be negligible.

6.12 BOND TRANSFER AND YIELD PENETRATION

The problem of bond deterioration in a beam-column joint undergoing severe reversed loading has been noted by researchers.[4,7] This was discussed briefly in Chapters one and three. Penetration of yield into the joint region effectively reduces the available development length over which a bar can transfer bond forces to the concrete of the truss shear resisting mechanisms. This will lead to bond stresses which could be greater than what the joint could sustain. Therefore substantial slippage of the bars may result.

In an elastic joint, no yield penetration is likely if the potential beam hinge is correctly designed. This means that the probable development length for beam bars would extend from the centroid of the column reinforcement in the tension face to the concrete compression face. The compression bar forces in the beam are lower than in the corresponding inelastic case because the beam concrete compression force at the joint boundary is maintained.

The aspect ratio may influence the bond transfer in an elastic joint. A reduction in the column width (i.e. smaller aspect ratio) may reduce the available development length to such an extent that the problem of bar slip may become critical.

6.13 DIAMETER OF BEAM AND COLUMN BARS

The diameter of beam and column bars is likely to influence the shear resistance of the joint. No study of this issue appears to have been made. Smaller bars are known to be more satisfactory for bond transfer than larger bars. A larger number of both beam and column bars of small diameter, spaced evenly across the respective beam and column faces at the joint, will probably lead to a more efficient introduction of shear stresses into the joint core than several large diameter beam or column bars. However, this practice would aggravate construction difficulties.

6.14 SPECIAL JOINT STEEL DEVICES

The possibility of using special joint steel devices has been investigated by some researchers[8] as a means of improving joint behaviour under inelastic actions. The use of a bond plate to transfer the shear forces directly to a diagonal concrete strut has been mentioned in section 1.2. Other proposals such as the utilisation of diagonal bars across the joint to transfer the shear forces have also been considered. Both concepts would probably involve greater fabrication costs and would not be necessary if the joint is 'protected' by potential plastic beam hinges located away from the column face.

CHAPTER SEVEN

COMPARISON OF RESULTS AND OBSERVATIONS

This chapter draws conclusions from the results presented in Chapter five, and critically examines the theoretical approach reviewed in Chapter three and utilised in Chapter six.

The elastic model uses two shear resisting mechanisms for the purpose of analysis. These are the diagonal concrete strut mechanism and the truss mechanism. The elastic model appears to be consistent with the observed behaviour of the two units tested.

The measured angle of cracking on the surface of the joint in the test units was similar to the predicted angle of the diagonal strut. The predicted angle was calculated from equilibrium of internal joint forces (see Appendix A). However, due to the variation of principal tensile stresses, which change as the beam load is increased, it does not follow that the observed crack inclination in the joints of the test units is the same as the inclination of the diagonal compression field.

The elastic model indicates how the increased axial load will result in a diagonal strut of greater width. This appears to be confirmed by the results of these tests. By observing the load-deflection relationships for the beams, given in Chapter five, it can be seen that the deflection in unit B2 at the top load of the elastic cycle was considerably less than in unit B1. As described in section 5.1.3, this appears to be due to the smaller joint distortions in unit B2. This suggests the joint has become stiffer. Indeed it is not too difficult to visualise that, of the two postulated shear resisting mechanisms of the elastic model, the one associated with a single diagonal corner to corner strut will be the stiffer one. In the truss mechanism tensile forces, resisted by horizontal and vertical joint shear reinforcement, are involved and these lead to relatively large tensile strains. This accounts for a relatively smaller stiffness of that system.

The extent to which the column axial load is able to provide a restraint in the vertical direction of the truss mechanism is unclear. With reference to Figs 3.1 and 3.3, it appears that the zone of compression at the boundary of the joint would be able to assist in resisting a proportion of the vertical component of the truss mechanism, V_{sv}. It is obvious that column reinforcement passing through the joint near the perimeter of the column section would not be as efficient in resisting shear as multilegged stirrup ties in the horizontal direction. The results of the tests are unable to indicate the proportion of the column bar force that comes from shear resistance in the joint.

The elastic model considers the simple force equilibrium of the internal forces at the joint. The method does not consider compatibility of deformations between the concrete strut and the truss mechanism. This may be significant in elastic joints. The concrete strut is the stiffer mechanism and if compatibility of deformations was considered, a greater proportion of the joint shear would be carried by it than a consideration of force equilibrium would suggest. It is evident that the diagonal concrete strut is a more efficient mechanism for joint shear transfer. This suggests that the elastic model would give a conservative estimate of the proportion of joint shear resisted by the two mechanisms. This appears to be verified by the results of the tests reported in Chapter five.

As the joint stirrup-ties were not fully instrumented, it is impossible to identify the exact magnitude of the total horizontal joint shear force resisted by the horizontal shear reinforcement. However, a reasonable estimate of the likely value of the horizontal shear carried by the joint shear reinforcement, V_{sh}, can be made from the strain readings measured on the outer ties, at the top load of a load cycle. In unit B1 after twelve cycles, the peak average strain in the outer joint ties was 53% of the yield strain. As a likely estimate it may be assumed that the inner ties are strained at 20% above the value of the outer ties. From the area and the measured yield strength of the tie, V_{sh} can be estimated (equation 3.4). The total horizontal joint shear, V_{jh}, is calculated in Appendix A for theoretical yield of the outer layer of beam reinforcement. The beam tip load at the maximum load of a cycle, P_i, was less than the theoretical yield load, P_y^*. Therefore the value of V_{sh} was increased by the ratio of P_y^*/P_i to give a value of V_{sh} which corresponds with the calculated value for V_{jh}. Using equation (3.6) the value for V_{ch} can then be obtained. In this, the mechanisms of dowel action and aggregate interlock have not been considered. By similar means, estimates of V_{ch} can be made for unit B2 at the high ($0.44\ f_c{}'A_g$) and at the intermediate axial loads ($0.25\ f_c{}'A_g$).

The proportions of the total joint horizontal shear resisted by the concrete mechanisms V_{ch}/V_{jh} so calculated are shown in Fig. 7.1 for the respective axial loads. It should be noted that in the evaluation of V_{sh}, the contribution of horizontal stirrups was deliberately overestimated in order not to overestimate the contribution of the concrete strut, V_{ch}. Higher stirrup strains in unit B2 under large axial compression may well have resulted from splitting of the concrete due to large diagonal compression, rather than from the demand on the truss mechanism for shear transfer.

For comparison, the suggested equation for V_{ch}, obtained from case studies using the elastic model (equation 6.1), is shown. This suggests that the elastic model is conservative in the low and intermediate column axial load regions. At the high axial load the elastic method appears unconservative, probably because of the large diagonal compression as discussed above.

It is interesting to compare the performance of unit B1 with Beckingsale's[7] unit B12 in the inelastic range. The strength of the joint of unit B12 was maintained so that plastic hinges formed in the beam at the column face. As described in section 4.4, the horizontal joint shear at the end of the test of unit B12 is very similar to that occurring at the elastic limit for unit B1. The beam tip load at the end of the test for unit B12, from an average of beam loads, was 145 kN. The beam loads applied at the elastic limit for unit B1 were 130 kN.

In unit B12 with 8 sets of 4×12.7 mm ties, only one tie was observed to yield at the end of the test after three cycles of ductility 6. The strength of the joint of unit B12 was maintained so that plastic hinges formed in the beam next to the column face. Near the top of the first cycle at ductility 6, severe slip of the top and bottom beam bars occurred through the joint. However, in unit B1 after only two cycles at a ductility of 4, all the ties had yielded and the average beam tip load dropped by 30% from its average maximum of 165 kN, reached in the first cycle at ductility 2. Pull-out of beam bars would have been an unlikely problem in units B1 and B2 because the joint was gradually failing in shear or shear compression and this led to rapid reduction of bond forces to subcritical levels.

To enable unit B1 to form satisfactory beam hinges at the column face, it is obvious that more than 8 sets of 4×12.7 mm ties would have been required. Calculations indicate that if a joint failure in unit B1 was to be prevented when similar overstrength from strain hardening of beam steel was to occur, 11 sets of 4 × 12.7 mm ties would have been required.

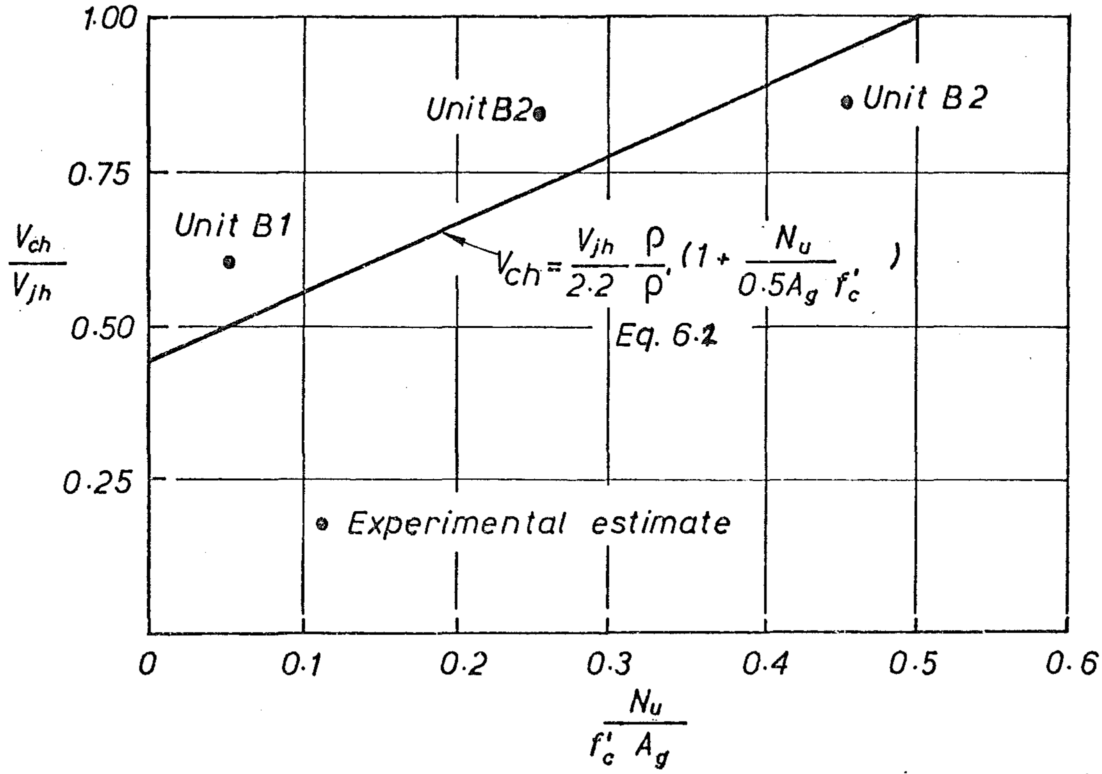


FIG. 7.1 COMPARISON OF EXPERIMENTAL ESTIMATE WITH ELASTIC THEORY

CHAPTER EIGHT

CONCLUSIONS AND RECOMMENDATIONS

Present knowledge of the shear behaviour of reinforced concrete beams suggests that sophisticated theories or analyses of reinforced concrete beam-column joints are unlikely to provide a satisfactory solution. The model postulated provides a simple approach to the analysis of the shear resisting mechanisms in elastic beam-column joints. By considering equilibrium of the internal forces acting at the joint, a proportion of the total joint shear force may be assigned to each of the two postulated shear resisting mechanisms. The results of the two tests indicate that this method of analysis should provide a satisfactory, conservative estimate of the required joint shear reinforcement in a cyclically loaded elastic joint. The test specimens performed well with respect to bond and pull-out of beam bars and the retention of shear strength under cyclic loading in the elastic range.

A full elastic analysis of a beam-column joint is not suggested for design. The present recommendations of the New Zealand National Society for Earthquake Engineering,[9] based on the concepts presented in the Paulay, Park and Priestley paper,[13] appear satisfactory for design purposes. A minor modification to the NZNSEE equation 2.18 could be made. Equation (6.2) is probably more realistic. Further extensions of the method of analysis for elastic beam-column joints could be made. However, it is not likely that this would lead to significant change in the joint shear reinforcement content. Practical considerations with respect to reinforcement layout would often override any adjustments resulting from refinements in analyses.

It is suggested that where relocated potential plastic beam hinges are employed, a considerable easing of previously encountered construction difficulties would ensue with a beam-column joint designed elastically. Indeed this is the reason for relocating plastic beam hinges away from column faces, a process inevitably involving some burden in fabricating the beam reinforcement. Unfortunately in many situations of seismic design, due to confinement restrictions, the construction benefits of an elastic joint may not be fully utilized. However, where the axial column compression load is small, confinement requirements should be less severe. Therefore an elastic beam-column joint with low axial load on the column will require less joint shear reinforcement than those joints where inelastic deformations occur in immediately adjacent members.

Similar advantages will result in elastic joints which may be considered to be confined by beams framing into the column from four sides, even though they may be carrying significant column compression loads.

The experimental program indicated that the test units, as designed, would be unsuitable for dissipation of energy resulting from severe seismic loading. It is thought that considerable post-elastic strength from other sources, such as dowel action of the column bars, was available in the beam-column joints tested.

The importance of considering vertical as well as horizontal shear has been emphasised. Future research may be able to provide more information on the role of vertical shear reinforcement, consisting of intermediate continuous column bars, in resisting joint shear forces.

APPENDIX A

DESIGN CALCULATIONS FOR UNITS B1 AND B2

Refer to drawings of test specimen (Fig. 4.2) for design constants and layout of reinforcement.

A.1 Beam

A.1.1 Properties: $f_y = 288$ MPa $\quad f_c' = 28$ MPa

modular ratio $n = \dfrac{E_s}{E_c} = \dfrac{2 \times 10^5}{4730\sqrt{28}} = 8.0$

A.1.2 Reinforcement: 8-D20 $\quad A_s = A_s' = 2510\ \text{mm}^2$

$\dfrac{d'}{d} = \dfrac{65}{545} = 0.119 \qquad \rho = \rho' = \dfrac{A_s}{bd} = \dfrac{2510}{356 \times 545} = 0.013$

A.1.3 Stresses due to M_y:

(tension +) $\quad k = [(\rho+\rho')^2 + 2(\rho+\rho'\dfrac{d'}{d})n]^{\frac{1}{2}} - (\rho+\rho')n = 0.317$

$\therefore\ kd = 173$ mm

$f_s = 275$ MPa $\quad f_c = \dfrac{k}{1-k}\dfrac{f_s}{n} = -15.9$ MPa $\quad f_s' = \dfrac{kd-d'}{kd} nf_c = -79.3$ MPa

$M = 0.5f_c\, bkd(d - \dfrac{kd}{3}) + f_s' A_s (d-d') = 334.0$ kN-m

Stresses in outer steel layer of top and bottom beam reinforcement.

$f_s = \dfrac{d-kd+20\text{mm}}{d-kd} \times 275 = 289.0$ MPa; $\quad f_s' = \dfrac{kd-d'+20\text{mm}}{kd-d'} \times 279.3 = -93.6$ MPa

A.1.4 Equilibrium of Forces

$$C_c + C_s = T$$

$$0.5\, f_c bkd + A_s'\ f_s' = A_s\ f_s$$

$$0.5 \times 15.9 \times 356 \times 173 + 2510 \times 79.3 = 2510 \times 275$$

$$490 + 200 = 690\ \text{kN}$$

A.1.5 Beam Shear:

at column face $M = V_b \left(\frac{4.876}{2} - \frac{0.457}{2}\right)$

$$334.0 \text{ kN-m} = V_b \times 2.21$$

$$\therefore V_b = 151 \text{ kN}$$

A.1.6 Ultimate Moment based on Steel Couple:

$$M_u = A_s f_y (d-d') = 2510 \times 288 \ (545-65) = 347.4 \text{ kN-m}$$

$$\therefore V_u = \frac{347.4}{2.21} = 157.2 \text{ kN}$$

A.2 Column

A.2.1 Properties: $f_y = 427$ MPa $f_c' = 28$ MPa $n = 8.0$

A.2.2 Reinforcement: 12-D24 $A_{st} = 5430 \text{ mm}^2$

$$\rho_t = \frac{A_{st}}{bh} = \frac{5430}{457 \times 457} = 0.026$$

A.2.3 Balanced load of Column:

(from design chart ACI Publication SP-7, "Ultimate Strength Design of Reinforced Concrete Columns", $\phi = 0.7$)

$$b = t = 457 \text{ mm}$$

$$g = \frac{363}{457} = 0.79 \qquad \rho_t = 0.026$$

$$m = \frac{f_y}{0.35\ f_c'} = 17.2 \qquad \rho_t m = 0.448$$

$$k_b = \frac{P_b}{f_c'\ bt} = 0.275 \qquad \therefore P_b = 1609 \text{ kN}$$

A.2.4 Ultimate Moment Capacity:

Unit B1 $P_u = 311 \text{ kN} = 0.19\ P_b$

$$\frac{P_u}{f_c'\ bh} = 0.053 \qquad \frac{P_u e}{f_c'\ bh^2} = 0.117$$

$$M_{uc} = 0.117 \times 28 \times 457^3 = 312.7 \text{ kN-m} \ (\phi = 0.7)$$

$$V_{col} = V_b \times \frac{4.876}{3.352}$$

assume $\phi_o = 1.25$ for beam $\therefore V_b = 1.25 \times 157.2 = 196.5$ kN

$$M_{col} = 196.5 \times \frac{4.876}{3.352} \left(\frac{3.352}{2} - \frac{0.61}{2}\right) = 391.9 \text{ kN-m}$$

using $\phi = 1.0$ $\quad M_{ic} = \frac{312.7}{0.7} = 446.7$ kN-m > 391.9.

<u>Unit B2</u> $\quad P_u = 2890 \text{ kN} = 1.8\ P_b$

$$\frac{P_u}{f_c'\,bh} = 0.494 \qquad \frac{P_u e}{f_c'\,bh^2} = 0.11$$

$$M_c = 0.11 \times 28 \times 457^3 = 294.0 \text{ kN-m} \quad (\phi = 0.7)$$

using $\phi = 1.0$ $\quad M_{ic} = \frac{294.0}{0.7} = 420.0$ kN-m > 391.9 kN-m

A.2.5 <u>Column Stresses:</u>

(At yield of beam tension reinforcement).

<u>Unit B1</u>

$$V_{col} = 151.2 \times \frac{4876}{3352} = 219.9 \text{ kN}$$

(at face of beam) $M_{col} = 219.9 \left(\frac{3.352}{2} - \frac{0.61}{2}\right) = 301.5$ kN-m

$$N = 311 \text{ kN}$$

by trial and adjustment the internal forces and neutral axis depth is found.

$$d = 457\text{-}47 = 410 \text{ mm}$$

try kd = 153.1 mm, $\quad f_c = -23.5$ MPa

$$f_{s1} = (457\text{-}47\text{-}153.1)\ \frac{23.5}{153.1} \times 8.0 = 315.5 \text{ MPa} \qquad T_1 = 571 \text{ kN}$$

$$f_{s2} = (457\text{-}147\text{-}153.1)\ \frac{23.5}{153.1} \times 8.0 = 192.7 \text{ MPa} \qquad T_2 = 174 \text{ kN}$$

$$f_{s3} = \frac{(153.1\text{-}147)}{153.1} \times 23.5 \times 8.0 = -7.5 \text{ MPa} \qquad T_3 = -7 \text{ kN}$$

$$f_{s4} = \frac{(153.1\text{-}47)}{153.1} \times 23.5 \times 8.0 = -130.3 \text{ MPa} \qquad C_4 = 236 \text{ kN}$$

$$C_c = 0.5\ f_c\ bkd = 822 \text{ kN}$$

equilibrium of forces: $\quad C_c + C_4 + C_3 = T_1 + T_2 + N$

$$822 + 236 + 7 = 174 + 571 + 311$$

$$1065 \simeq 1056 \text{ kN}$$

$$M = N.e = \left(\frac{457}{2} - 47\right) 571 + \left(\frac{457}{2} - 147\right) 174$$

$$+ 822 \left(\frac{457}{2} - \frac{153}{3}\right) + 236 \left(\frac{457}{2} - 47\right)$$

$$+ 7 \left(\frac{457}{2} - 147\right)$$

$$= 306 \text{ kN-m} \quad (\text{cf. } 301.5)$$

Unit B2

$$M_{col} = 301.5 \text{ kN-m}$$

$$N = 2890 \text{ kN}$$

$$d = 457-47 = 410 \text{ mm}$$

$$\text{try} \quad kd = 401.2 \text{ mm}, \qquad f_c = -26.2 \text{ MPa}$$

$$f_{s1} = (457-47-401.2) \frac{26.2}{401.2} \times 8.0 = 4.6 \text{ MPa} \qquad T_1 = 8 \text{ kN}$$

$$f_{s2} = (401.2-163-147) \frac{26.2}{401.2} \times 8.0 = -47.7 \text{ MPa} \qquad C_2 = -43 \text{ kN}$$

$$f_{s3} = (401.2-147) \frac{26.2}{401.2} \times 8.0 = -132.8 \text{ MPa} \qquad C_3 = -120 \text{ kN}$$

$$f_{s4} = (401.2-47) \frac{26.2}{401.2} \times 8.0 = -185.0 \text{ MPa} \qquad C_4 = -335 \text{ kN}$$

$$C_c = 0.5 f_c bkd = 2402 \text{ kN}$$

equilibrium of forces: $C_c + C_4 + C_3 + C_2 = T_1 + N$

$$2402 + 335 + 120 + 43 = 8 + 2390$$

$$2900 \simeq 2898 \text{ kN}$$

moments about column centre.

$$M = N.e = \left(\frac{457}{2} - 47\right) 8 - \left(\frac{457}{2} - 147\right) 43 + \left(\frac{457}{2} - 147\right) 120$$

$$+ \left(\frac{457}{2} - 47\right) 335 + \left(\frac{457}{2} - \frac{401.2}{2}\right) 2402$$

$$= 300 \text{ kN-m} \quad (\text{cf. } 301.5)$$

A.3 Joint Shear Reinforcement

Unit B1

A.3.1 Properties: $f_y = 345$ MPa for 12.7 mm ties

$f_y = 427$ MPa for D24 bars.

A.3.2 Horizontal Joint Shear:

(beam load $V_b = 151$ kN)

$$V_{jh} = T + C_s + C_c - V_{col} = 690 + 200 + 490 - 220 = 1160 \text{ kN}$$

$$V_{ch} = \Delta T_c + C_c - V_{col}, \qquad \Delta T_c = \gamma\,(C_s + T)$$

$$\text{assume } \gamma = 3/4\,\frac{kd_{col}}{d} = 3/4\,k_{col} = 3/4 \times \frac{153.1}{410} = 0.28$$

$$\Delta T_c = 0.28\,(690 + 220) = 249.3 \text{ kN}$$

$$V_{ch} = 249.3 + 490 - 220 = 519 \text{ kN}$$

$$\therefore V_{sh} = V_{jh} - V_{ch} = 1160 - 519 = 641 \text{ kN}$$

$$\tan\beta = \frac{822 + 243 + 745 - 151}{490 + 200 + 690 - 200}$$

$$= 1.43$$

$$\beta = 55^{\circ}$$

from joint geometry

$$\tan\beta = \frac{610}{457} = 1.33$$

$$\beta = 53^{\circ}$$

A.3.3 Vertical Joint Shear:

$$V_{cv} = V_{ch}\,\tan\beta = 519 \times 1.43 = 742 \text{ kN}$$

$$V_{sv} = V_{sh}\,\tan\beta = 641 \times 1.43 = 917 \text{ kN}$$

A.3.4 Horizontal Joint Shear Reinforcements:

4 legs of 12.7 mm ties $\quad A_{jh} = (12.7)^2\,\frac{\pi}{4} \times 4 = 507 \text{ mm}^2$

$$n = \frac{V_{sh}}{A_{jh}\,f_y} = \frac{558 \times 10^3}{507 \times 345} = 3.2$$

use 4 sets of 4 × 12.7 mm ties.

A.3.5 Vertical Joint Shear Reinforcement:

D24 bars $\quad A_{jv} = 452 \text{ mm}^2$

$$n = \frac{V_{jv}}{A_{jv}\,f_y} = \frac{917 \times 10^3}{452 \times 427} = 4.7$$

The above calculations ignore the axial compression on the column section. As explained in section 4.4.2 and section 6.6, there are difficulties in designing the amount of vertical shear reinforcement required. It is suggested that the intermediate bars shown in Fig. 4.2 are adequate.

Unit B2

A.3.6 Properties: $f_y = 398$ MPa for 6.5 mm ties

$f_y = 427$ MPa for D24 bars

A.3.7 Horizontal Joint Shear:

(beam load $V_b = 151$ kN)

$V_{jh} = 1160$ kN as before.

assume $\gamma = 3/4\ k_{col} = 3/4 \times \dfrac{401.2}{410} = 0.73$

$\Delta T_c = 0.73\ (690 + 200) = 653.2$ kN

$V_{ch} = 653.2 + 490 - 220 = 923$ kN

$\therefore\ V_{sh} = V_{jh} - V_{ch} = 1160 - 923 = 237$ kN

$$\tan\beta = \frac{2402 + 498 + 8 - 151}{490 + 200 + 690 - 220}$$

$$= 2.38$$

$$\beta = 67^{o}$$

A.3.8 Vertical Joint Shear;

$V_{cv} = V_{ch} \tan\beta = 923 \times 2.38 = 2197$ kN

$V_{sv} = V_{sh} \tan\beta = 237 \times 2.38 = 564$ kN

A.3.9 Horizontal Joint Shear Reinforcement:

4 legs of 6.5 mm ties $A_{jh} = (6.5)^2 \times \frac{\pi}{4} \times 4$

$= 133\ mm^2$

$$n = \frac{V_{sh}}{A_{jh} f_y} = \frac{237 \times 10^3}{133 \times 398} = 4.5$$

4 sets of 4 × 6.5 mm ties were used.

A.3.10 Vertical Joint Shear Reinforcement:

D24 $A_{jv} = 452\ mm^2$

$$n = \frac{V_{jv}}{A_{jv} f_y} = \frac{564 \times 10^3}{452 \times 427} = 2.9$$

This ignores the substantial axial compression on the column section.

APPENDIX B

DEVELOPMENT BOND

B.1 Beam Steel:

From ACI code[3] requirements the development length required for a bar in tension is

$$f_y = 288 \text{ MPa}, \quad f_c' = 28 \text{ MPa}, \quad l_d = \frac{0.019\, A_b f_y}{\sqrt{f_c'}} = 325 \text{ mm} \qquad \text{D20 bar}$$

for a bar in compression

$$l_d = \frac{0.241\, f_y d_b}{\sqrt{f_c'}} = 262 \text{ mm} \qquad \text{D20 bar}$$

from the elastic analysis at $P_e = 130$ kN.

$$f_s = 248.5 \text{ MPa}, \qquad f_s' = -80.5 \text{ MPa}$$

It is assumed that the bar stress changes linearly from tension at one end to compression at the other as indicated in Fig. 5.9.

$$\therefore \quad l_d = h_c - d' = 457 - 47 = 410 \text{ mm}$$

where h_c = depth of the column, and
d' = depth to the centroid of the column bars nearest the beam.

Associated bond stress from the theoretical stresses.

$$u = \frac{d_b(f_s + f_s')}{4.\, l_d} = \frac{20\ (248.5 + 80.5)}{4 \times 410} = 4.0 \text{ MPa}$$

allowable development length from ACI code.[3]

$$l_d = \frac{248.5}{288} \times 325 + \frac{80.5}{283} \times 262 = 354 \text{ mm}$$

associated permissible bond stress

$$u = \frac{20(248.5 + 80.5)}{4 \times 354} = 4.7 \text{ MPa}$$

B.1.1 Unit B1

(from Fig. 5.9) at cycle 15 $\mu = 4$

f_s = 300 MPa gauge location 7

f_s' = -260 MPa gauge location 4

the stress transfer is approximately linear between these two points.

Maximum possible development length ld = 400 mm

associated bond stress $u = \dfrac{20\ (300 + 260)}{4 \times 300} = 9.3$ MPa

B.1.2 Unit B2

(from Fig.5.12) at cycle 23 $\mu = 4$

f_s = 288 MPa gauge location 7

f_s' = -130 MPa gauge location 3

the stress transfer is approximately linear between these two points.

Maximum possible development length ld = 400 mm

associated bond stress $u = \dfrac{20\ (288 + 130)}{4 \times 400} = 5.0$ MPa.

Classn:

ELASTIC BEHAVIOUR OF EARTHQUAKE RESISTANT
REINFORCED CONCRETE INTERIOR BEAM-COLUMN JOINTS

G. Birss

ABSTRACT: Results are presented of tests of two interior beam-column joints under simulated earthquake loading. The elastic behaviour was investigated. The units were then tested until failure. An elastic model was reviewed and was found to provide a satisfactory and conservative estimate of joint shear reinforcement required in an elastic joint. The response of these test units was found to be unsatisfactory in the post-elastic range.

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